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Saturation Flow Adjustment Factors for Right Turning Traffic at Signalized Intersections

by

Miro Poss

A thesis
submitted to the Faculty of Graduate Studies and Research
in partial fulfilment of the requirements for the degree of

Master of Science

Department of Civil Engineering

Edmonton, Alberta

Spring 1985

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ABSTRACT

This thesis deals with right turning vehicular flows influenced by pedestrians at signalized intersections.

In urban areas the frequent crossing of roads (intersections) is the most important factor influencing the operation of the whole system. Intersections reduce the vehicle flow in the street network. In many cases traffic control signals must be provided to maintain adequate system capacity. Especially in downtown areas, where pedestrian flows are heavy, traffic cannot be handled without proper signalization and correct timing. For this reason an evaluation of each intersection is necessary. Similarly as in many other cases of interference with vehicle flows, the concept of saturation flow is used for the analysis.

At the present time there are three methods available in Canada for evaluation of right turning vehicular flow with pedestrian interference (Toronto, Edmonton and Yagar's). A major task of this project was to test the three methods in Edmonton conditions and to select the most suitable one for further development.

The goal of this research was to design a technique which would include all important variables involved in the right turning problem. A procedure was suggested and verified in Edmonton conditions. It includes such factors as: pedestrian flows, geometric conditions (especially turning radius and length of the crosswalk), signal timing

conditions (length of green and walk intervals) as well as some other conditions (such as traffic pressure).

The suggested method for the analysis of the right turning problem was found to produce satisfactory results. Moreover, although it is theoretically sound, it is simple enough to be understood and used by the practicing traffic engineers.

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CHAPTER 1 - INTRODUCTION

1.1. Research Problem

Although much has been written about roadway capacity, the need to evaluate the factors determining the vehicle flow capacity at intersections still exists. This thesis deals with the problem of right turning traffic flow at signalized intersections. The concept of saturation flow (Refs. 1 to 10) was employed and attempts were made to define the adjustment factor variations for the different cases encountered.

Traffic engineers utilize the Capacity Guide (Ref. 8) for evaluation and design. The recently published Canadian Capacity Guide for Signalized Intersections quantifies all factors which influence intersection capacity. The Guide is based on Canadian experience and many portions have already been verified. There is however a need for more detailed investigation of the right turning problem.

Three potentially available methods were developed in Canada and can be used for evaluation of the right turning vehicular flow. These are:

A. The method used in the Toronto Intersection Capacity Guide (Ref. 9). This procedure was derived from research done in Australia. It was modified and applied to Toronto conditions. The relationship between pedestrians and vehicles is expressed by a linear regression line.

B. The empirical graph (function) developed by the University of Alberta from the data obtained in Edmonton (Ref. 8). This method could be used as a practical and a simple tool (function) for traffic engineers to evaluate right turning flows at the signalized intersection. The carefully observed conditions typical for individual locations are also included.

C. The method suggested by Professor Yagar - The University of Waterloo (Ref. 10). This includes mathematical expression of probabilities of the right turning vehicles penetrating the pedestrian flow under specific circumstances. Detailed verification was carried out in traffic conditions in Waterloo and Kitchener.

In the case of this research, the three methods were applied to the Edmonton conditions and the theoretical results were compared with the measured data.

Emphasis has been made on the practicality and accuracy of these methods, and the results obtained are presented in an easily understandable form.

1.2. Arrangement of the Thesis

The thesis consists of nine chapters and four appendices. The contents of the following chapters are briefly outlined below.

1.2.1 Foreword

The foreword contains the abstract, acknowledgement, list of contents, list of figures, list of tables and list of references.

1.2.2 Chapter 1 - Introduction

This part deals with introduction to the problem undertaken in the thesis and its arrangement.

1.2.3 Chapter 2 - Review

This part deals with a brief introduction to the saturation flow theory, description of right turns and possible situations at the signalized intersection. The statement of the objectives and the scope of the thesis is also presented.

1.2.4 Chapter 3 - Description of the Models

This part briefly describes all the three available methods (Toronto, Edmonton, and Yagar's) which are presently used in Canada for evaluation of right turning vehiclular flow with pedestrian interference.

1.2.5 Chapter 4 - Data Collection

This part contains a decision of the data collection process and comparison of the three methods. Each method uses different input data. General format of data

collection suitable for all three methods is described, survey locations identified and survey techniques presented.

1.2.6 Chapter 5 - Comparison of the Three Methods

Chapter 5 is concentrated on evaluation of the three methods. This part contains also a brief introduction to the data preparation for comparison of the available methods. Systems of data files and simple programs are mentioned. The method used for evaluation is described and results are graphically presented. Findings are sumarized for each method and conclusions drawn from comparison are presented.

1.2.7 Chapter 6 - Further Development of Edmonton Method

Computer programs used in this part are briefly mentioned and the method used for development is described step by step together with presentation of partial results. The basic formula is introduced and important limitations are verified. The necessity of applying geometric and timing factors is proven.

1.2.8 Chapter 7 - Effect of Turning Radius and Timing

The fit of basic formula is examined for locations with different conditions. The most important adjustment factors are recognized and their influence is studied.

1.2.9 Chapter 8 - Composite Evaluation

The most important adjustment factors are evaluated and a trial and error method is used for the development of a final formula. Statistical evaluation of the results is included. Presentation is provided also graphically and a concept for evaluation is formulated.

1.2.10 Chapter 9 - Conclusions

Conclusions drawn from this research are summarized. They are presented for each step of research.

1.2.11 Appendices

Appendices contain the supporting documents, computer programs, preliminary results, tables, graphs and descriptions. The printouts of the programs are also included in this part.

CHAPTER 2 - REVIEW

2.1. Problem statement

The right turning movement of the vehicle flow at signalized intersections in many cases penetrates pedestrian flow at the crosswalks. Interaction between vehicles and pedestrians reduces the capacity of the intersection and consequently has an impact on the whole road network. In the downtown areas, because of higher pedestrian and vehicular flow, the situation is more critical. Hence, for the purpose of design and analysis a knowledge of the correct values of maximum discharge rate is essential.

2.2. Saturation flow concept

In recent times, the saturation flow concept for the evaluation of urban transportation networks has been used across Canada (Refs. 8 to 11).

Saturation flow is affected by many factors. Basic Saturation Flow is defined as the number of passenger car units which can discharge across the stopline of an "ideal" intersection approach lane moving straight, without additional traffic friction under ideal weather conditions and during an optimum length of green interval. The concept is shown in Figure 2.3. Basic Saturation Flow reflects only the ideal geometric, traffic or weather conditions.

Initial Saturation Flow values can be determined for the specific conditions in a community or region. They

depend on weather conditions and intersection environment.

In the calculation of the basic saturation flow the appropriate adjustment factors have to be taken into account.

A detailed description of the saturation flow concept is presented by the Canadian Capacity Guide (Ref. 8). The Guide was recently published and includes illustrative examples of calculations.

Stability of flow over a long period of time is shown in Figure 2.1. Saturation flow is expressed in passenger car units per hour of green (pcu/h green).

Saturation flow is generally expressed on a cumulative basis, according to the time after green was started (Fig. 2.2). The true values for the general location can be seen from Figure 2.3. 1550 pcu/h green is used as the basic saturation flow for the City of Edmonton.

2.2.1 Adjustment factors

To calculate a saturation flow value for specific conditions the initial saturation flow of the lane is used. Generally these conditions require adjustments. They are as follows:

A. Geometric factors

   a. lane width

   b. gradient

   c. turning radius

   d. queueing and discharge space

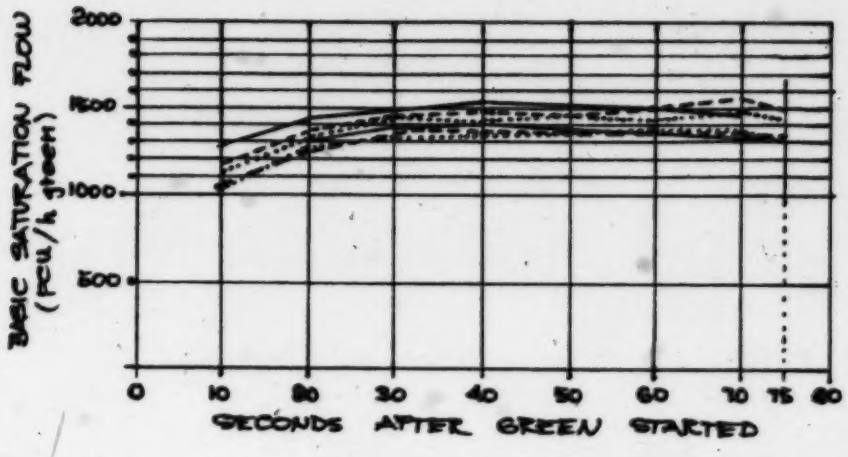


Figure 2.1 Saturation flow surveys (by the University of Alberta) represented by envelopes of partial results. For details see Appendix 1, Fig. A.1.

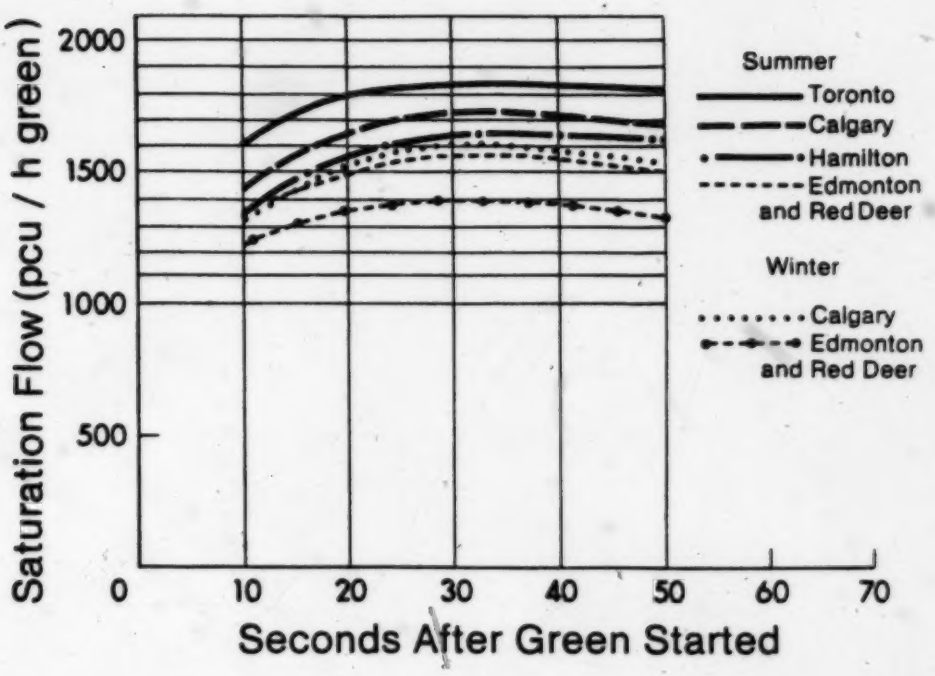


Figure 2.2 Typical ranges of Basic Saturation Flow (from Ref. 8). The values of the flow are based on cumulative form of expression and can not be directly compared to Figure 2.3. (no copyrights)

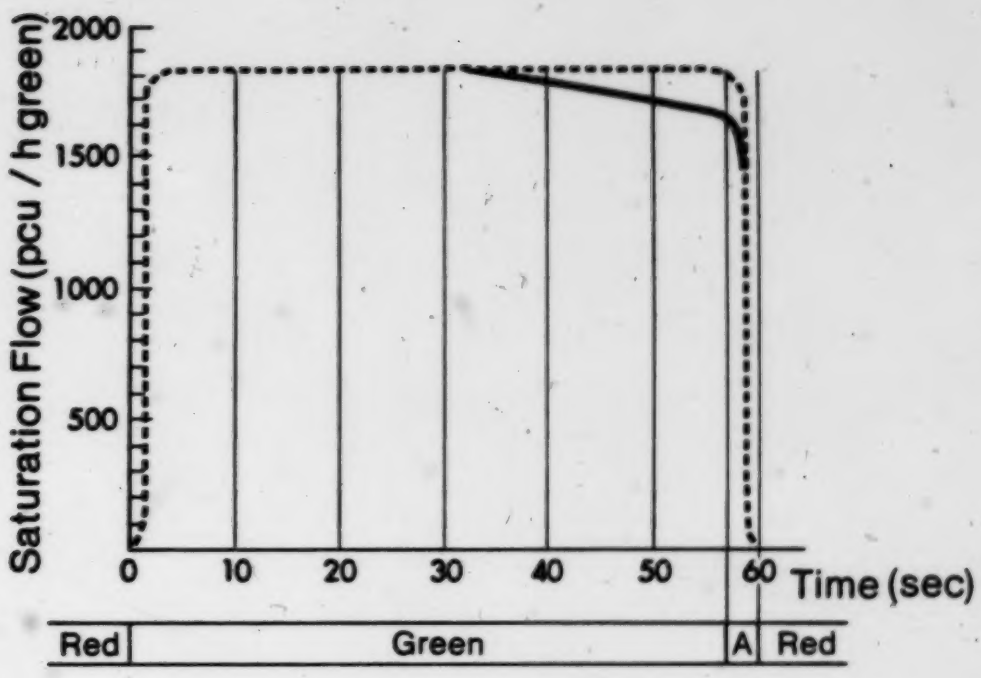


Figure 2.3 The saturation flow concept (from Ref. 8). The broken line shows the traditional concept, the solid line is based on Canadian research. (no copyrights)

B. Traffic condition factors

a. public transit

- nearside stops
- farside stops
- bus/trolley stops
- streetcar stops
- exclusive bus lane function

b. parking interference

c. pedestrians

d. length of green

e. flow rates in other lanes (i.e. opposing flow)

C. Other factors

a. weather conditions

b. illumination

c. psychological pressure

If there is more than one condition which calls for adjustments, the combination of adjustment factors must be carefully considered (Table 2-1) since they may not be multiplicative.

2.3. Objectives of the thesis

This study was mainly concerned with the problem of right turning vehicles penetrating pedestrian flow. The objectives of the research were:

A. identification of variables influencing the capacity of right turning vehicular movements as affected by

Lane Function	Possible Adjustments							
	Lane Width	Radius	Grade	Queueing Space	Discharge Space	Bus Stops	On-street Parking	Length of Green
Straight Through and Left	•		•	•	•	•	•	•
Straight Through and Right	•	•	•	•	•	•	•	•
Straight Through Only	•		•	•	•	•	•	•
Left Only	•		•	•		•		•
Right Only	•	•	•	•		•		•
Straight Right and Left	•	•	•	•	•	•	•	•

Table 2-1 Possible combinations of adjustment factors to the initial saturation flow in relation to different lane functions (Ref. 8). (no copyrights)

pedestrians.

B. comparison of existing methods and determination of their practical usage.

C. determination of adjusted saturation flow for right turn vehicle flow penetrating pedestrian flow at the adjacent crossing.

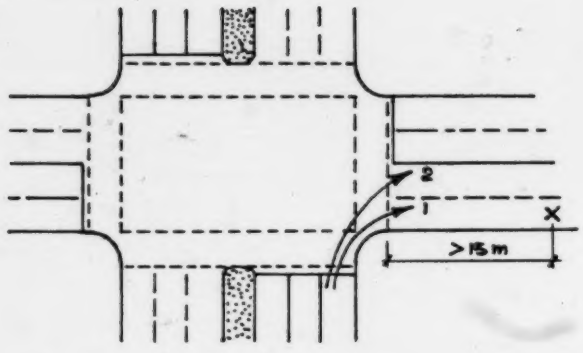
2.4. The right turning problem

2.4.1 Usual signal phasing

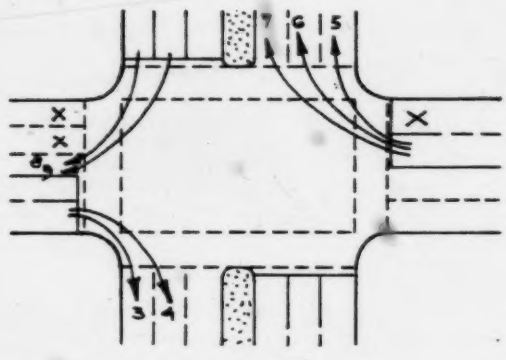
A "typical" urban signalized intersection has an at-grade pedestrian crosswalk which is an interaction area between right turning vehicles and pedestrians. Pedestrian flow is the most critical influencing factor.

Because of other usual intersection constraints, the complete separation of pedestrians from vehicular flow in time or space cannot generally be provided. In most cases pedestrians and vehicles share the same phase. Because right turning vehicles must yield to pedestrians, they must find adequate gaps between pedestrians for completion of their turning maneuver. This interaction brings about significant friction and consequently reduces the capacity. Figure 2.4 shows a "typical" geometric configuration. Figure 2.5 shows the situation in the interaction zone for different timing stages.

Pedestrians approach the interaction zone from two sides of the street. They interact with vehicle flow as groups (platoons) or at random. Figure 2.5 illustrates



a. TYPICAL CONDITIONS



b. LESS USUAL CONDITIONS

1,2 are typical movements of right turning vehicles

3,4 movements can be considered atypical for downtown locations because of the width of the street crossed by pedestrians.

5,6 movement is considered atypical for downtown locations because of the obstruction in two inner lanes and bigger interaction zone.

7,8,9 right turning vehicle flow from second approach lane can be found in downtown areas, but it is also considered atypical for this research case.

X obstruction of lane (parking, construction work, congestion, etc.)

Figure 2.4 Geometrical configurations for right turning lanes - typical according to the thesis subject.

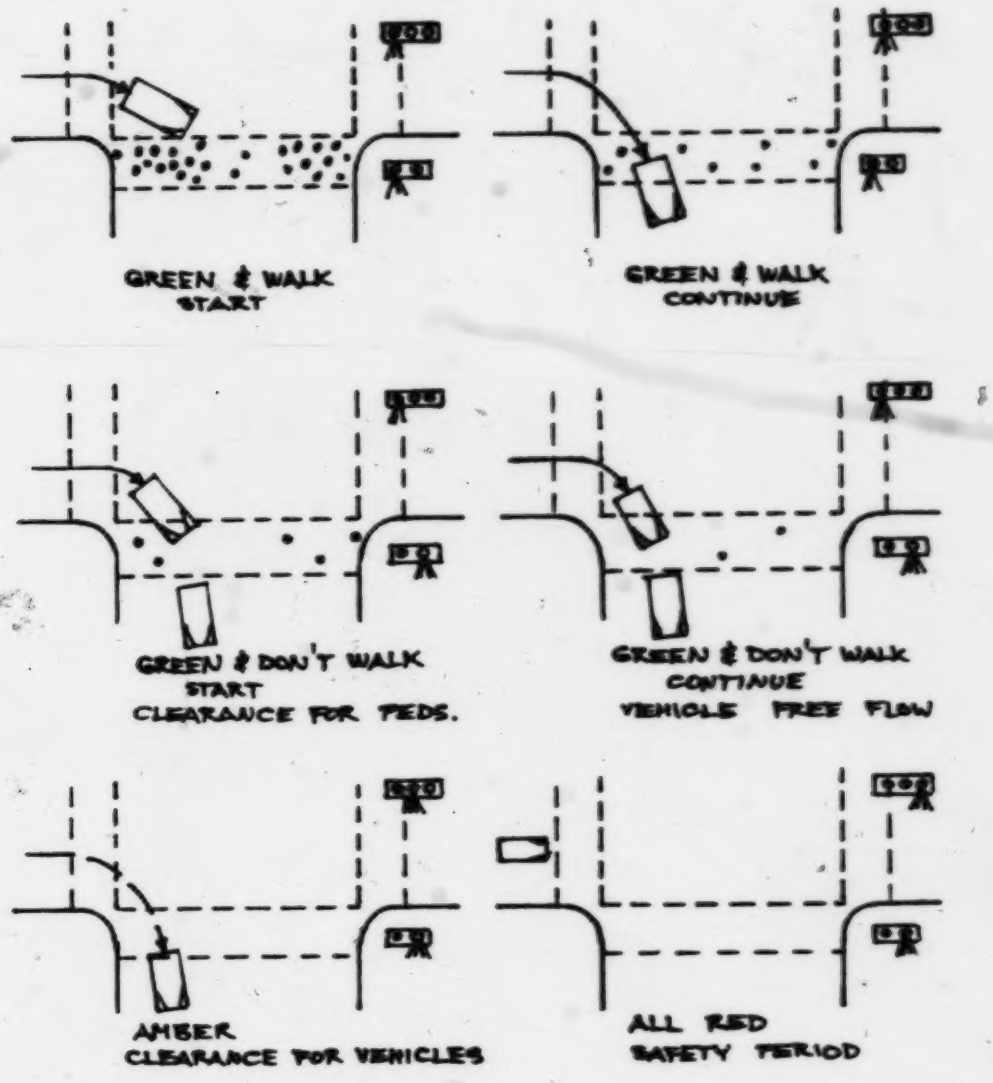


Figure 2.5 Vehicles and pedestrians interaction during one phase of the cycle.

possible situations including the frequent disobedience of pedestrians as well as the vehicles turning during the intergreen phase.

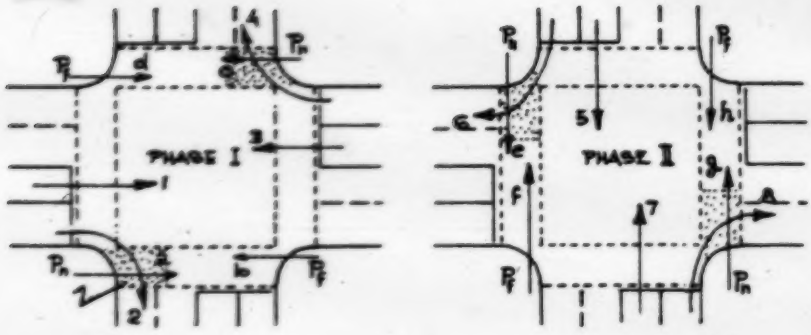
As a consequence of the space-sharing nature of traffic signal operation, there are safety considerations which form inviolable constraints.

Minimum "walk" interval is defined as a minimum of a "walk" signal display during which the group of pedestrians who were waiting for this display can reach a reasonable distance in the crosswalk at a comfortable speed.

A pedestrian "clearance period" is defined as the time required for the pedestrian who entered the crosswalk at a very end of the "walk" interval to reach a safe refuge.

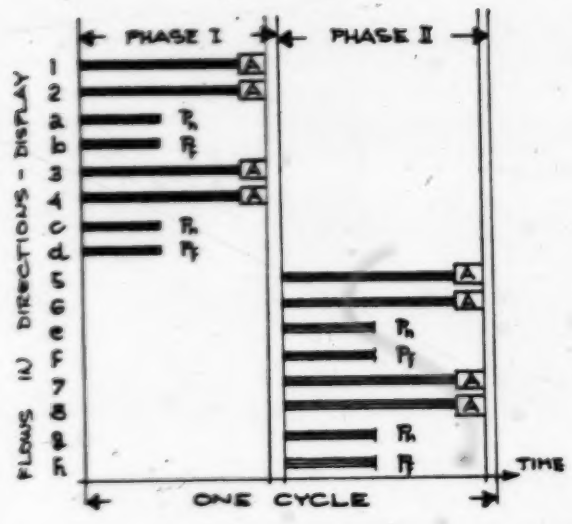
Geometrical configuration, timing diagram and the magnitude of the flows are shown in Figures 2.6 to 2.8. For practical use, the flow diagram is simplified as shown in Figure 2.9. The magnitudes of vehicle and pedestrian flow rates in the figure are exaggerated to emphasize initial and rest periods of vehicles penetrating the interaction zone (Z).

Morning, afternoon and lunch-time peaks yield high flows of vehicles and pedestrians. Their interaction causes problems in downtown areas. Adequate timing is necessary to reduce delays and discomfort for both pedestrians and drivers.



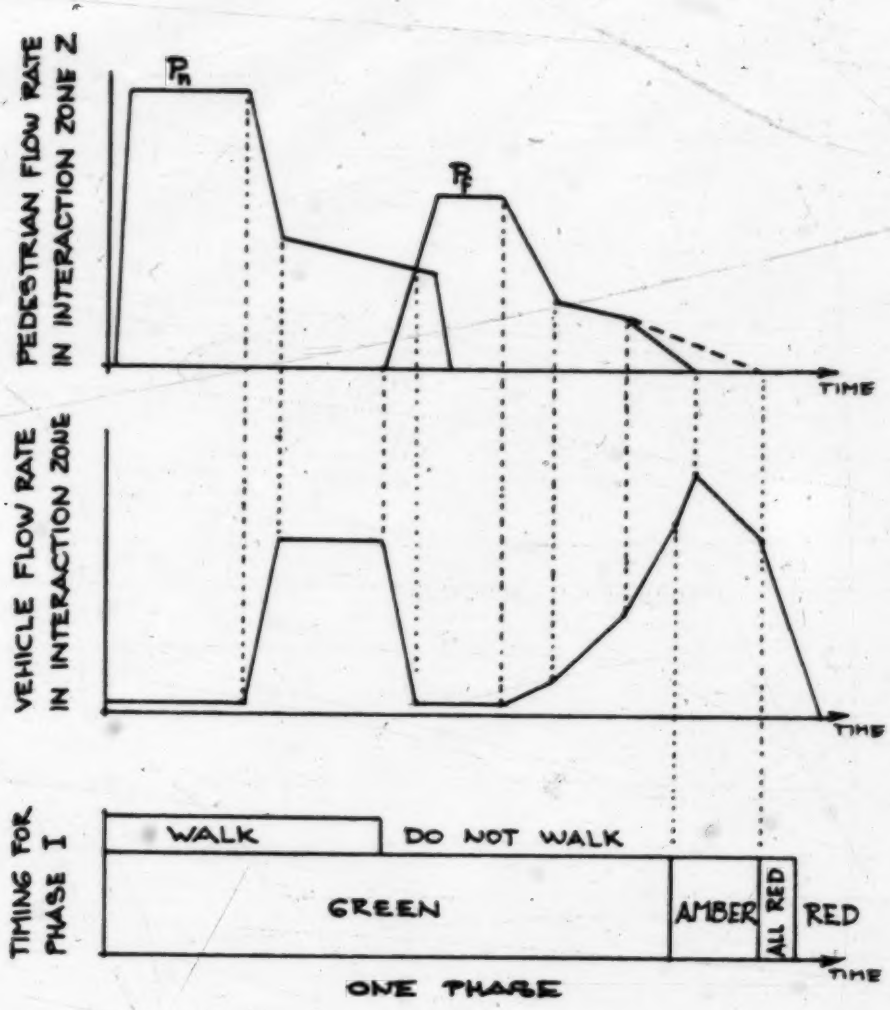
▒ - interaction zones
P_n - pedestrian flow from near side
P_f - pedestrian flow from far side

Figure 2.6 Pedestrian and vehicle flows at the intersection with two phase timing.



■ - green light displayed on console
△ - amber
□ - all red interval for all directions

Figure 2.7 Timing schemes - display for lanes and crosswalks



P_n - pedestrian flow from near side

P_f - pedestrain flow from far side

Figure 2.8 Flow diagrams for vehicles and pedestrians interacting in crosswalk zone Z.

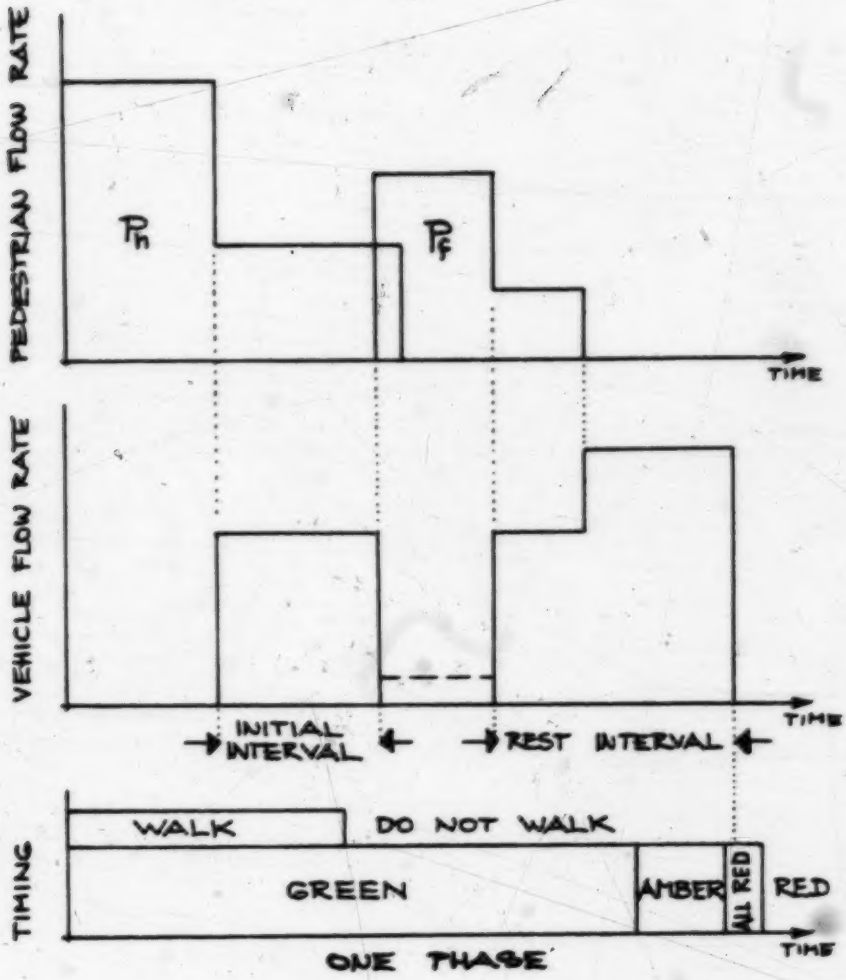


Figure 2.9 Simplified flow diagram for one phase of the cycle.

2.4.2 Signal phasing separating vehicular and pedestrian flows

Pedestrian and vehicle flows can be separated from each other. Two kinds of separation can be provided.

A. Physical separation can be provided by over- or underpasses or skyways. Examples can be found in Minneapolis, Spokane or Calgary.

B. Partial or total timing separation can also be provided.

2.4.2.1 Signal phasing with partial separation of vehicular and pedestrian flows.

As an example, partial timing separation for common geometrical conditions is used in Vancouver (Fig. 2.10). Vehicles receive several seconds of green interval in advance of the beginning of walk interval. This period is provided for a free flow of right turning vehicles. After the beginning of walk interval the usual interaction of vehicles and pedestrians takes place.

2.4.2.2 Signal phasing with total separation of vehicular and pedestrian flows

Total timing separation is handled in different ways by using the advanced phases of the cycle exclusively for pedestrians (walk phase) and for vehicles.

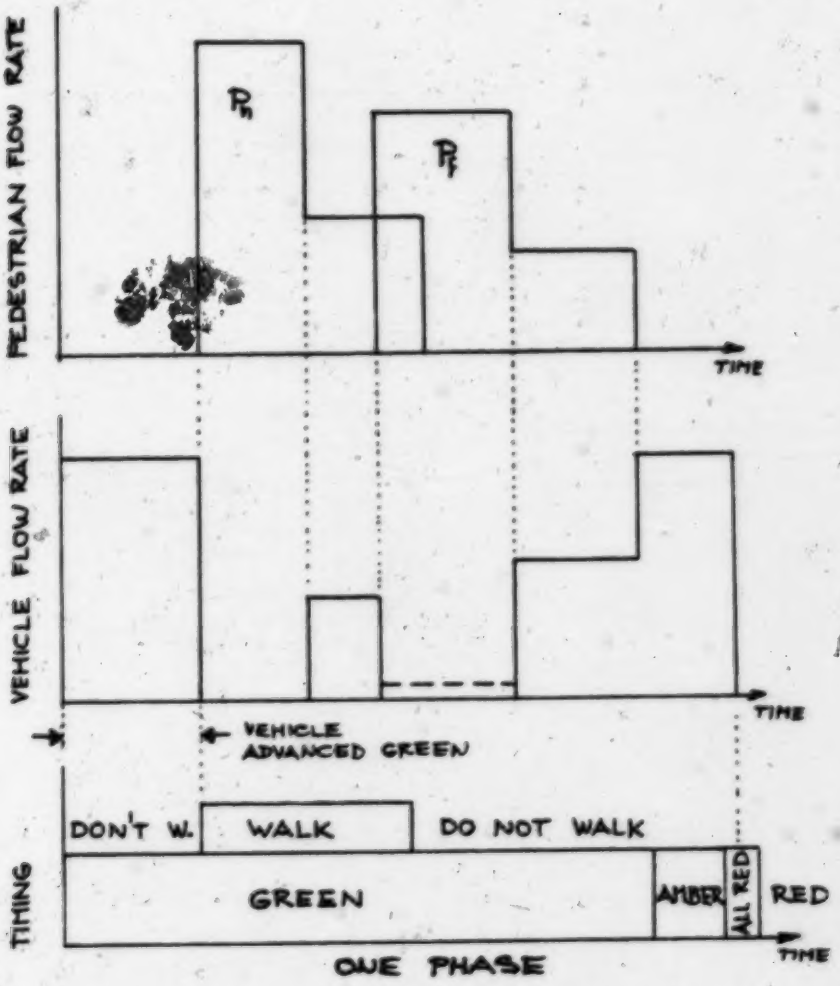


Figure 2.10 Example of partial timing separation. The geometric configuration is the same as in Figure 2.6.

A. The advantage for vehicles

The practical and usual arrangement for total timing separation is shown in Figure 2.11. In this timing scheme (Fig. 2.12) the geometrical arrangements are also involved, and such a solution is used when one flow, mainly vehicle flow, is significantly higher than the others.

B. The advantage for pedestrians

The following arrangements are listed for completeness of this report. Such solutions to this problem are not used very often in North America.

C. Separate signal phasing for pedestrian and vehicular flows

The "scrambled" scheme (Fig. 2.13) presents a less usual arrangement, i.e. as in Seattle, Washington, D.C. etc. This timing scheme (Fig. 2.14) is exclusively used for heavy pedestrian flows usually for narrow cross streets. Using the scheme for wide streets is somewhat impractical due to cycle time requirements. A major feature of this solution is that one phase of the cycle is used exclusively for pedestrian flows typically for considerations with heavy pedestrian flows and minor vehicular flows such as during lunch peaks.

D. Special arrangements

Another example of total separation was implemented in

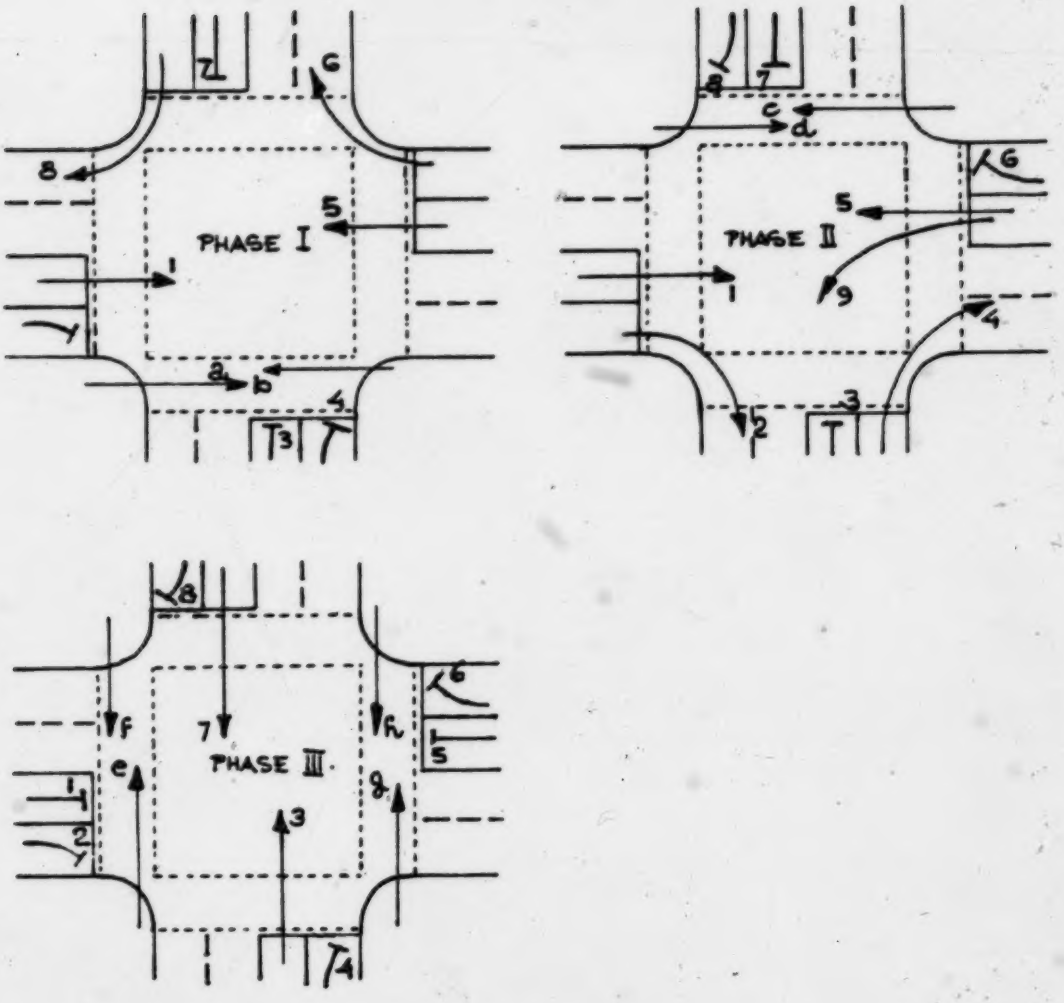


Figure 2.11 Example of total timing separation - geometrical arrangement. This arrangement is advantageous in the case of high vehicle flow.

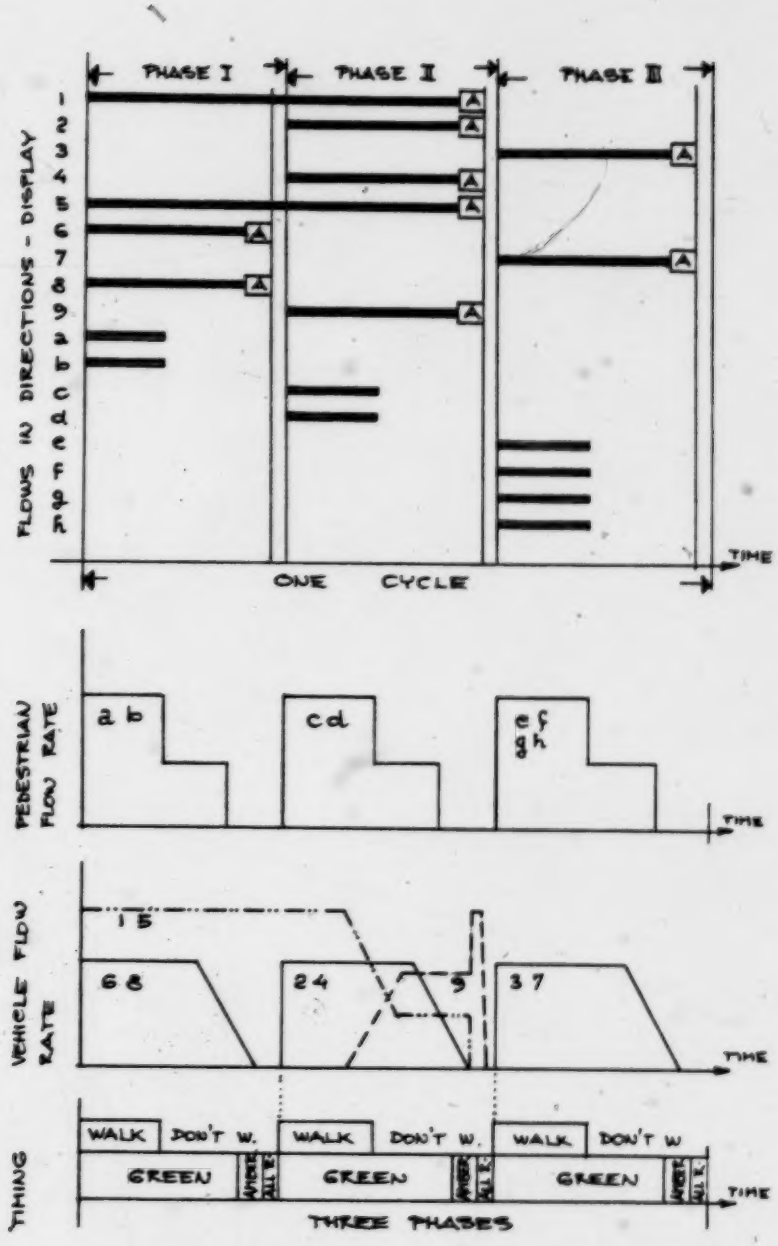


Figure 2.12 Total timing separation - timing diagram and flow chart.

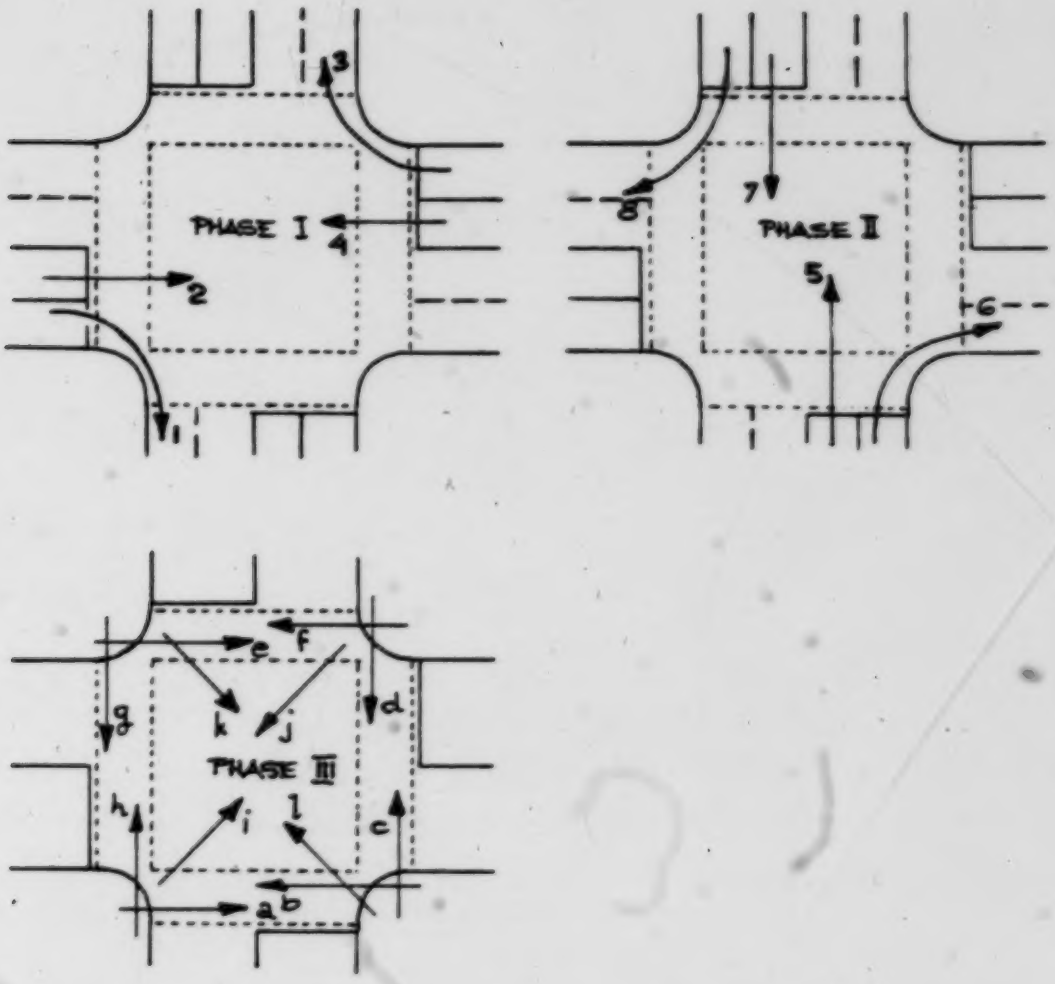


Figure 2.13 Example of intersection with scrambled timing - geometrical arrangement.

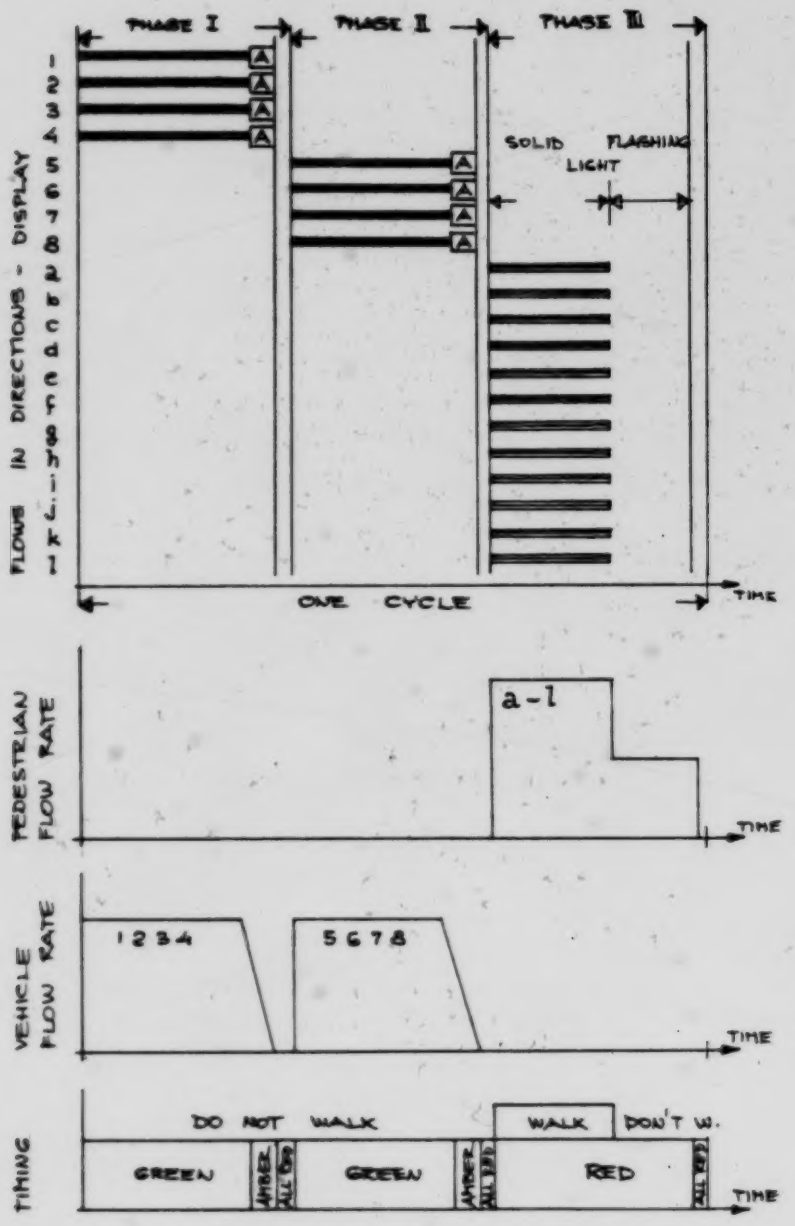


Figure 2.14 Intersection with scrambled timing - timing diagram and flow chart.

Duesseldorf, West Germany. A portion of the downtown area is closed during the lunch time peak to all vehicles. The high pedestrian flows have absolute freedom of movement during lunch hours.

In some areas only supply and maintenance vehicles are granted entry to the downtown. In such a case, they have to yield to pedestrians. Examples can be found in cities with pedestrian precincts (zones).

2.5. Scope of research

In organizing this research project a number of factors were defined to meet research objectives. The following considerations assisted in limiting the research to a manageable level.

A. Only the right turns during green intervals for typical urban conditions were studied.

B. This study was concerned with a practical approach, that is a development or verification of procedure which can enhance the techniques used in the first edition of Canadian Capacity Guide. The objective was to develop a tool for use to practicing professionals rather than researchers.

C. Only three models will be compared (Toronto, Edmonton and Yagar's model).

CHAPTER 3 - DESCRIPTION OF MODELS

For evaluation of right turning vehicular flow penetrating the pedestrian flow, presently there are three models available in Canada. They are as follows:

3.1. Toronto model

Toronto Traffic Control Centre uses a modified and calibrated Australian model (Ref. 9, 12, 13, 14 to 17). The model has a form of:

$$S = 1073 - 0.21P \qquad (1)$$

where S - saturation flow value (passenger car units/h green)

P - pedestrian flow (pedestrians/h green)

Figure 3.1 shows its graphical presentation. Vehicles are converted to passenger car units (pcu). A pcu equivalent of 2.25 pcu/vehicle is used for all vehicles with dual tires.

The saturation flow for exclusive right turn lanes at signalized intersections is expressed as a function of pedestrian flow rate on the adjacent crosswalk regardless of the sidewalk from which pedestrians approach. Since pedestrians approach the crosswalk during an hour but only a portion of cycle time can be used for their discharge, the volume must be converted into flow rate of pedestrians per

hour green. It is calculated as:

$$P = V_{ped} \frac{c}{g} \qquad (2)$$

where P = converted pedestrian volume (pedestrian flow rate)(ped/h green)

V_{ped} = pedestrian volume (ped/h)

c = cycle time (sec)

g = green interval (sec)

The advantage of the Toronto method is in its simple expression and application to quick evaluation of right turns. The formula is recommended by the Canadian Capacity Guide for Signalized Intersections as one of the two methods for evaluation of right turning vehicle flow penetrating pedestrian flows (Ref. 8).

3.2. Edmonton model

The University of Alberta's contribution to the analysis of right turns has been by development of a new function. A pilot study in Edmonton (1982) indicated that a nonlinear function better reflects the conditions than the linear one which is used in Toronto. Similarity to opposed left turns was found for the relationship of pedestrians versus right turning vehicular flow.

The survey technique applied to the pilot project was similar to that used for other saturation flow surveys. The

cycle time was divided into 10 second "slices". Collection of data was provided for each "slice". Complete description is presented in Reference 5.

Both the mathematical and graphical form was used for regression. Data from a pilot project were presented in the form of polynomial lines without a mathematical equation. These results were published in Guide Draft #4 (Ref. 11). The basic ideas of the Edmonton approach were adopted and developed in this research. The exponential form derived from the first phase of this project was recommended by The Canadian Capacity Guide as one of the two methods for evaluation of right turns. Both forms are presented in Figure 3.2.

3.3. Yagar's model

Yagar suggested an analytical model for evaluation of right turn movement with interference of pedestrians. The model is a modified version of the Swedish procedure (Ref. 9) which is used also in Australia. This method has not been published however the paper is available at The University of Waterloo (Ref. 10).

The model distinguishes between the pedestrians approaching the interaction zone from nearside and farside curb (Fig. 2.9). The pedestrians interaction with vehicles is therefore divided into two parts which are evaluated separately.

Yagar's model assumes that pedestrians arrive randomly

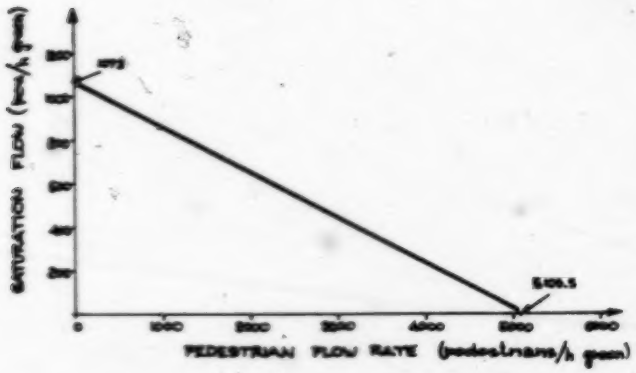


Figure 3.1 Right turn saturation flow with pedestrian interference - Toronto method

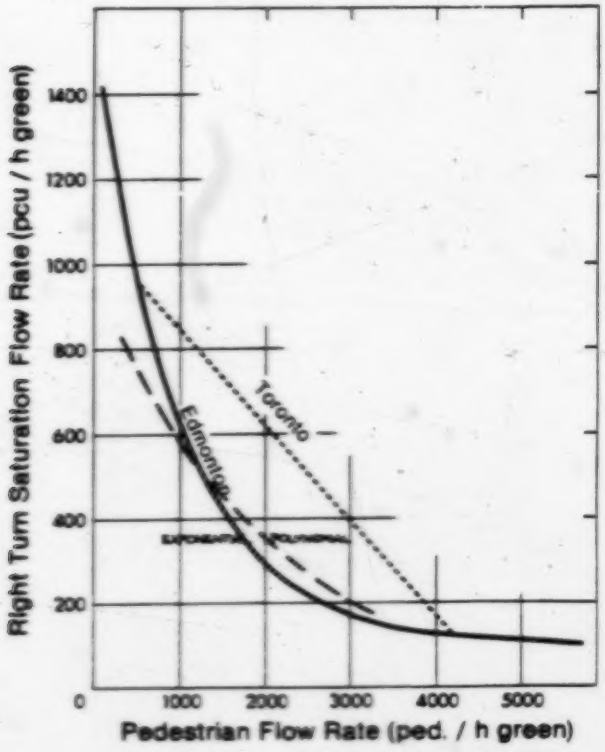


Figure 3.2 Edmonton method in polynomial and exponential form. (adapted from Ref. 8)

according to the Poisson distribution. The survey carried out in Toronto and Kitchener has shown that pedestrians arrive in groups with the average size of 1.7 ped/group. Since a group of pedestrians typically arrive together and walk side by side, the group is taken as a single pedestrian. Although pedestrians arrive at random, they normally queue for the green phase. Therefore they create nearside and farside platoons served at the beginning of the green period, followed by random arrival from both sides. The arrival rates are denoted by P_n for nearside and by P_f for farside pedestrians (in pedestrians per hour). The situation in the interaction zone is presented in Figure 2.9.

The average walking speed of pedestrians (S) was found to be 1.2 m/sec. The model also approximates the queue of pedestrians as a single file with service headways (H) equal to 0.45 sec.

The width of the street (W) crossed by pedestrians determines the offset betwen the arrivals of nearside (P_n) and farside (P_f) pedestrians into the interaction zone (Fig. 3.3 and 2.9). The farside pedestrians begin arriving $(W-W_c)/S$ seconds after the nearside pedestrians. This affects gap availability.

The driver must find a gap between the pedestrians which is large enough for him to accelerate and drive through the crosswalk without interfereing with the last pedestrian before him or the first one after him. Predicted

required initial gap was 5 sec. The gap extension for each additional vehicle was 3.6 sec.

The calculated numbers of right turning vehicles depends upon the structure of the signal cycle, specifically nongreen time (R) and the cycle time (c). The value of turning flow can be treated as saturation flow, since the right turns on red are not included.

Yagar's model developed for right turn capacity is presented in Figure 3.4.

The initial time (I) which is required to clear both initial queues of pedestrians is calculated. It is the greater of times required for the nearside and farside queues to clear the interaction zone. These times are represented by t_1-R and t_2-R in the following expressions (Fig. 3.5).

Taking W_c = 3.6 m, S = 1.2 m/sec, H = 0.45 sec

$$I = \max \left(\frac{R + \frac{12}{4}}{1 - 0.45\ P_n} - R \right) ; \left(\frac{R + \frac{W}{4}}{1 - 0.45\ P_f} - R \right) \qquad (3)$$

Usually $t_2 > t_1$ (the opposite can be found in the case of a shift of lights, etc.).

There are three possible cases: A. where queues overlay in the intersection zone, B. where they must meet and C. where there is a time gap between them. In the last case some vehicles can be served through gaps in the random nearside pedestrian stream after the initial nearside queue has been served. These are denoted as "Initial" vehicles.

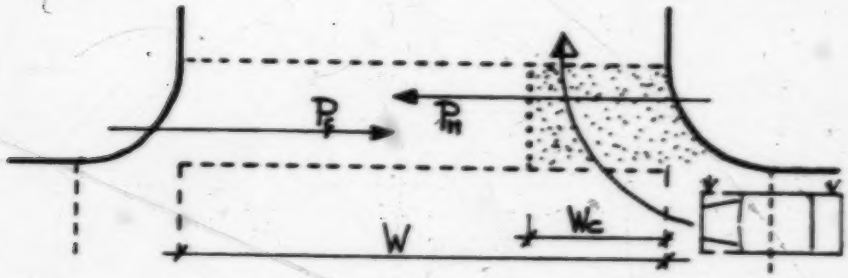


Figure 3.3 Situation at the interaction zone - by Yagar (Ref. 10). (no copyrights)

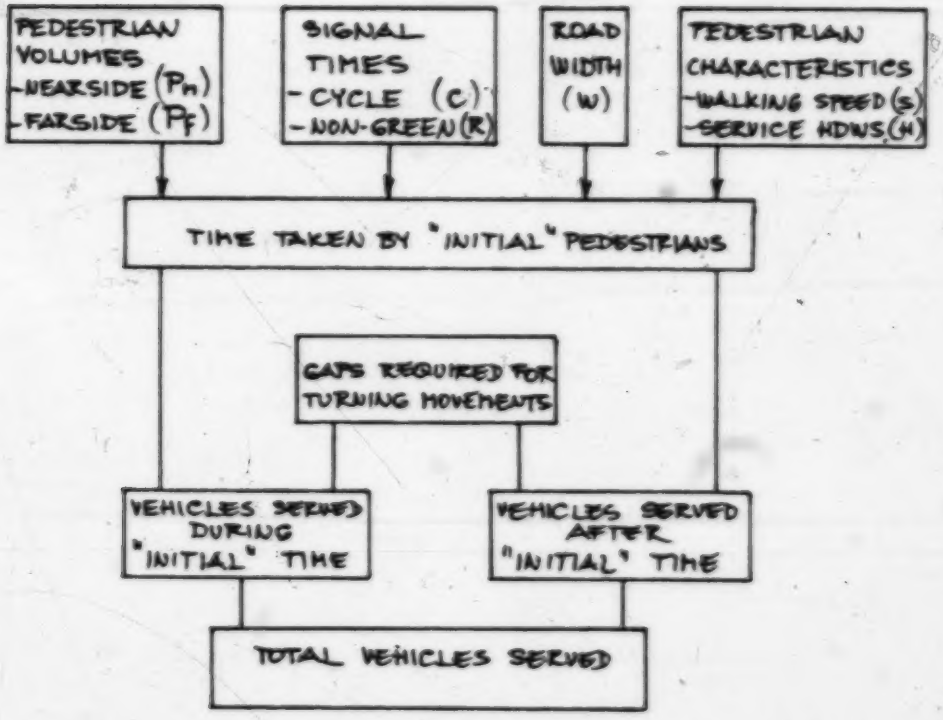
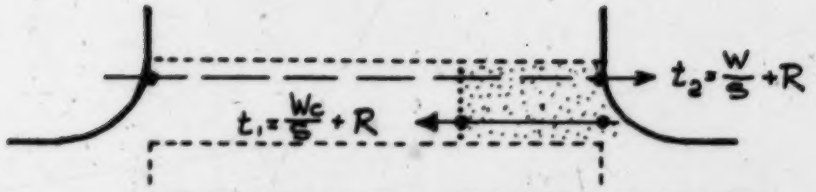
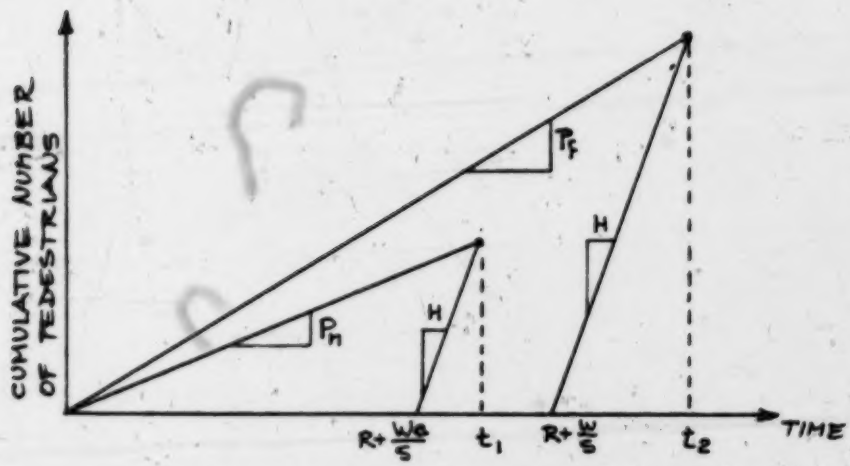


Figure 3.4 Steps for calculation of right turning capacity. (no copyrights)



- EXPRESSION OF THE REQUIRED TIMES AT THE INTERACTION ZONE



- CUMULATIVE PEDESTRIAN ARRIVALS AND SERVICE

where:
- P_n = pedestrian arrival rate - nearside
- P_f = pedestrian arrival rate - farside
- R = red + amber time
- W = road width
- W_c = distance from nearside curb to the end of intersection zone
- H = service headway for queued pedestrians
- S = pedestrian walking speed
- I = initial time
- I = max $(t_1; t_2)$ - R

Figure 3.5 The time required to reach and clear the interaction zone (from Ref. 10). (no copyrights)

In all cases some vehicles may find gaps in the combined random pedestrian streams during the "Rest" of green phase (Fig. 2.9).

The number of right turning vehicles for the "Initial" and the "Rest" period is calculated as follows. The initial gap is calculated by the formula

$$G_I = \frac{W - W_C}{S} - \frac{R + \frac{W_C}{S}}{1 - H P_n} + R \qquad (4)$$

If the gap size is $t^* > 5$ sec a right turning vehicle is served. For each additional 3.6 sec of t^* another vehicle is served and the average number of vehicles is a sum of the following probabilities

$$P(t^* > 5) + P(t^* > 8.6) + P(t^* > 12.2) + \text{etc.} \qquad (5)$$

The arrival rate during the initial time is P_n (ped/sec) for individual pedestrians or $P_n/1.7$ (groups/sec) for grouped pedestrians. Arrival headways are exponentially distributed and therefore capacity ($^*C$) for right turns through an initial gap is approximately given as follows:

$$^*C_I = 0 \qquad \text{for } G_I < 5 \qquad (6)$$

$$^*C_I = e^{-5P_n/1.7} \qquad \text{for } 5 < G_I < 8.6 \qquad (7)$$

$$^*C_I = e^{-5P_n/1.7 + e^{-8.6P_n/1.7}} \qquad \text{for } 8.6 < G_I < 12.2 \qquad (8)$$

etc.

$$*C_I = \sum_j e^{-(5+3.6j)P_n/1.7} \qquad \text{for } 5+3.6j < G_I < 8.6 + 3.6j \quad (9)$$

Capacity of the total initial gap is

$$*C_I{}^T = C_I \max\left(1;\ P_n/1.7G_1\right) \quad (10)$$

The vehicles are also served after the queues of pedestrians have cleared the crosswalk. The remainder of the cycle time, during which vehicles may find gaps is

$$G_R = G - I_{max} \quad (11)$$

and the pedestrian arrival rate is

$$P_n + P_f \text{ or } P_n + P_f/1.7 \text{ (groups/sec)} \quad (12)$$

The number of vehicles served per gap is

$$*C_R = 0 \qquad \text{for } G_R < 0 \quad (13)$$

$$*C_R = \sum_j d^{-(5+3.6j)\frac{P_n+P_f}{1.7}} \qquad \text{for } 5+3.6j < G_R < 8.6 + 3.6j \quad (14)$$

Capacity of the total rest gap is

$$*C_R^T = 1 \qquad \text{for } G_R < 5 \qquad (15)$$

$$*C_R^T = 1 + \frac{P_n+P_f}{1.7} G_R \sum_j e^{-(5+3.6j)\frac{P_n+P_f}{1.7}}$$

$$\text{for } 5+3.6j < G_R < 8.6 + 3.6j \qquad (16)$$

Yagar's factors as an initial and rest period are also observed for the Edmonton working format. This fact provides an advantage for direct comparison of real values with Yagar's theoretical results.

CHAPTER 4 - DATA COLLECTION

4.1. Data requirements

A comparison of the existing methods was the first task to be undertaken in this study. The methods were described in the previous chapter. Each of them required a different form of data input and therefore a survey and data collection technique which would be suitable for all of them should be devised.

4.1.1 Toronto method

The Toronto method required the input of pedestrian flow rate (in pedestrians/hour of green). Its flow rate was calculated from the number of nearside and farside pedestrians entering the interaction zone. The green interval only was used as the calculation time base. The number of pedestrians interacting during green interval was used as input. The formula for calculation was described in Chapter 3. No additional information was needed.

4.1.2 Edmonton method

The input for the Edmonton method also required that the number of observed pedestrians and vehicles be converted into flow rates expressed in units per hour of green. However the observations were carried out for base intervals with increments of 10 seconds. According to this method, the total pedestrian as well as vehicle volumes interacting

in 10 second "slices" of the cycle time were converted into the units per hour of green by using the formula:

$$P' = \frac{\Sigma N'}{T'} * 3600 \qquad N' = N'_n + N'_f \tag{17}$$

where P' = pedestrian flow rate (in ped/h green) for desired time "slice"

T' = time of observed "slice" (in seconds) in dependence of accumulation type: 10 sec, 20 sec, 30 sec, 40 sec and 50 sec.

$\Sigma N'$ = number of pedestrians crossing the intersection zone from nearside (N'_n) and from farside (N'_f) during the time T'.

Vehicle flow rates were calculated in a similar way as pedestrians. The number of pedestrians and vehicles for each 10 sec "slice" was required for the evaluation of this method and was used as input data.

4.1.3 Yagar's method

Yagar's method required information about geometric and timing conditions. Required data are:

A. length of crosswalk

B. length of interaction zone

C. cycle time

D. green interval.

The method assumes that pedestrians are in the crosswalk

during walk interval and clearance period. The sum of both two periods is equated with duration of green interval. Some of the basic parameters used in the model were verified for Edmonton conditions in a simplified way. Those are as follows:

A. pedestrian speed

B. pedestrian grouping factor

C. vehicle acceleration

D. average headways between pedestrian groups

E. probabilities of gap acceptance

F. arrival pattern

No significant deviation was found and these factors were used as presented in Reference 10.

Distinction for the number of pedestrians starting from the nearside and farside curb had to be made. Since this technique is based on arrival rate of pedestrians at the crosswalk, the counts had to be taken as presenting hourly arrival volumes. Pedestrian counts were converted to hourly flows by using the formula:

$$P_n \text{ or } P_f = \frac{\Sigma N_n' \text{ or } \Sigma N_f'}{c} \cdot 3600 \qquad (18)$$

where $P_n; P_f$ = pedestrian flows from nearside or farside (in pedestrians per hour)

$N_n'; N_f'$ = number of pedestrians crossing the interaction zone from nearside or farside during green time G

c = cycle time

The number of pedestrians during the green interval was used as input, similarly to Toronto method.

4.1.4 Data to be collected

For the surveys the following field information was gathered:

A. Intersection geometry
   a. situation
   b. width of lanes
   c. width of crosswalks
   d. width of streets
   e. dimensions of interaction zone
   f. turning radius

B. Traffic signal setting
   a. cycle time
   b. amber interval
   c. green interval
   d. pedestrian walk time
   e. red interval

C. Pedestrians
   a. walking speed
   b. headways
   c. magnitude of groups
   d. behaviour
   e. number of pedestrians crossing the interaction

zone in 10 second intervals from both sides separately

D. Vehicles

a. speed

b. acceleration

c. headways

d. gap requirements

e. drivers behaviour

f. influence of obstructions

g. influence of shared lanes

h. number of vehicles entering the intersection zone in 10 second intervals

4.1.5 Limitation on data sampling

The basic data required for all three methods were as follows:

A. street width (w)

B. interaction zone length (w_c)

C. cycle time (c)

D. green interval (G)

E. pedestrian flows from both sides of the crossed street (P_n;P_f) penetrating vehicles (K) and straight through going vehicles (KS), all recorded in 10 second slices

F. pedestrian walk (g) was recorded for the later determination of the "free vehicle flow".

4.2. Location

4.2.1 Suitability of location

Not every intersection was suitable for our purpose. A final selection was based on preliminary observation in the CBD (Central Business District or Downtown) and a few locations were chosen. The requirements were as follows:

A. moderate to high pedestrian flows

B. moderate to high vehicle flows

C. simple geometry (lanes functioning as exclusive for right turns)

D. no obstructions on approach and discharge lanes

E. turning radius typical for CBD

F. variable timing conditions

G. no grade

4.2.2 Choice of locations

Set of locations was selected according to the requirements from the previous paragraph. In each case the direction (for example east bound) gives the location of the crosswalk. They were:

Location #1: Jasper Ave.-101 St., east bound. This intersection provided an exclusive lane for right turns with occasional use by straight through vehicles and limited number of turning buses. Curb radius was 8.5 m which is considered to be typical for CBD conditions.

Timing conditions during surveys, identical for lunch and afternoon peaks were: cycle time 80 sec, green interval 39 sec, amber 3 sec, walk interval 29 sec. Observations were carried out under summer conditions with dry pavement. 280 cycles were recorded, 118 cycles were used for evaluation, about 7800 pedestrians and 1400 vehicles were involved. A variety of conditions was observed.

Location #2: 102 Ave.-100 St., south bound. The intersection provided an exclusive lane for right turns with occasional use by straight through and turning buses. Almost no other straight through traffic was recorded. Curb radius was 8.5 m. Timing conditions for lunch peak were: cycle time 80 sec, green interval 33 sec, walk interval 18 sec. Timing conditions for afternoon peak were: cycle time 90 sec, green interval 40 sec, walk interval 25 sec. Amber interval was 3 sec in both cases. Observations were carried out under summer conditions with dry and wet pavement. 285 cycles were recorded, 209 cycles were used for evaluation, about 4400 pedestrians and 1430 vehicles were involved. No special conditions were observed.

Location #3: Jasper Ave.-101 St., south bound. The intersection provided an exclusive lane for right turns with occasional straight through bus traffic. Curb radius was 8.5 m. Timing conditions for morning peak were: cycle time 80 sec, green interval 31 sec, walk interval 23 sec. Timing conditions for lunch peak were: cycle time 90 sec, green interval 40 sec, walk interval 31 sec. Amber interval was 3 sec in both cases. Observations were carried out under summer conditions with dry pavement. 205 cycles were recorded, 125 cycles used for evaluation, about 10 000 pedestrians and 820 vehicles were involved. This location yielded high pedestrian flows during the University Games. The cycles with an obstructed approach lane were excluded from evaluation.

Location #4: 102A Ave.-100 St., west bound. This intersection provided an exclusive lane for right turns with occassional straight through bus traffic. Curb radius was 8.5 m. Timing conditions for lunch peak were: cycle time 80 sec, green interval 22 sec, walk interval 13 sec, amber interval 3 sec. Afternoon peaks were not observed. Observations were carried

out under summer conditions with dry pavement. 55 cycles were recorded, 47 cycles were used for evaluation, about 1100 pedestrians and 180 vehicles were involved. The right turns were influenced in some cases by left turns from opposite direction at the beginning of phase. High degree of platooning was observed and sensitivity of vehicle flow to the total number of pedestrians was low.

Location #5: Jasper Ave.-102 St., east bound. The intersection provided an exclusive lane for right turns with occasional straight through bus traffic. Curb radius was 8.5 m. Timing conditions for afternoon peak were: cycle time 80 sec, green interval 31 sec, walk interval 20 sec, amber interval 3 sec. Morning and lunch peaks were not observed because of inadequate vehicle flows. Observations were carried out under summer conditions with dry pavement. 78 cycles were recorded, 32 cycles were used for evaluation, about 1100 pedestrians and 360 vehicles were involved. In some cases the right turns were influenced by left turns from opposing direction at the end of the phase.

Location #6: 102A Ave.-101 St., north bound. This intersection was chosen for its exclusive lane for right turns with radius of 15.5 m. Timing conditions were: cycle time 80 sec, green interval 30 sec, walk interval 22 sec, amber interval 3 sec. Observations were carried out under summer conditions with dry pavement. Over 100 cycles were observed, 45 cycles were recorded, 13 cycles included in evaluation, about 270 pedestrains and 230 vehicles were involved. This location had low pedestrian activity. No unusual conditions were observed.

Location #7: 82 Ave.-109 St., south bound. This intersection provided an exclusive lane for right turns with occasional straight through bus traffic. Curb turning radius was observed to be 13.0 m due to the utilization of third discharge lane. Timing conditions were: cycle time 80 sec, green interval 30 sec, walk interval 20 sec, amber interval 3 sec. Observations were carried out under summer conditions with dry pavement. 95 cycles were recorded, 76 cycles were evaluated, about 865 pedestrians and 475 vehicles were involved. The respect for pedestrians was higher than in other locations. In some cases left turns from

opposing direction slightly influenced right turns during the phase.

Summary of locations is presented in Table 4-1 and individual sketches of geometric conditions for researched locations are presented in Figures 4.1 to 4.6.

4.2.3 Typical location

For comparing of the three methods, as well as the suitability of used data and format, a test of the procedure was needed. Location #2 was selected as a "typical" one because of its traffic and geometric conditions. As well data were available from pilot project 1981-82. This location was examined with regard to changes in signal timing. At any time of the day there were high vehicle and pedestrian flows. Morning, lunch time and afternoon peaks were selected as the most suitable day time periods.

4.2.4 Timing and geometric factors

Locations with different geometric conditions were compared and the influence of turning radius was studied. Different timing conditions were also evaluated separately.

4.2.5 Special conditions

The effect of "The World University Games 1983" event on traffic conditions was also exploited in this research. The cumulation of people during the games provided extremely

#	Location	Timing Cycle/Gr. /Walk	Radius/ Width of Crossed Street	Traffic Conditions
1	Jasper Ave. 101 St. E-Bound	80/39/24	8.5/20.2 Δ 15	Exclusive right turn lane for vehicles. Turn lane used by straight through vehicles. Exceptionally turning buses.
2	102 Ave. 100 St. S-Bound	90/40/25 80/33/18	8.5/19.4 Δ 15	Exclusive right turn lane. Turning buses. Straight through traffic - buses only.
3	Jasper Ave. 101 St. S-Bound	90/40/31 80/31/23	8.5/13.7 Δ 9	Exclusive right turn lane. Occasional limited obstruc. No turning buses. Straight through traffic - buses only.
4	102A Ave. 100 St. W-bound	80/22/13	8.5/17.1 Δ 9	Exclusive right turn lane. Opposing left turns at the beginning of green. Minimum straight through buses. No turning buses.
5	Jasper Ave. 102 St. E-Bound	80/31/20	8.5/21.5 Δ 11	Exclusive right turn lane. Opposing left turns at the end of phase. Straight through buses. No turning buses.
6	102A Ave. 101 St. N-Bound	80/30/22	15.5/17.3 Δ 8	Exclusive right turn lane. No buses. No straight through traffic. Low traffic pressure.
7	82 Ave. 104 St. S-Bound	80/30/20	13.0/22.0 Δ 10	Exclusive right turn lane. Opposing left turns during phase. Straight through buses well saturated flows. Lower traffic pressure.

*Note: Δ - Green minus Walk difference

Table 4-1 Description of observed locations

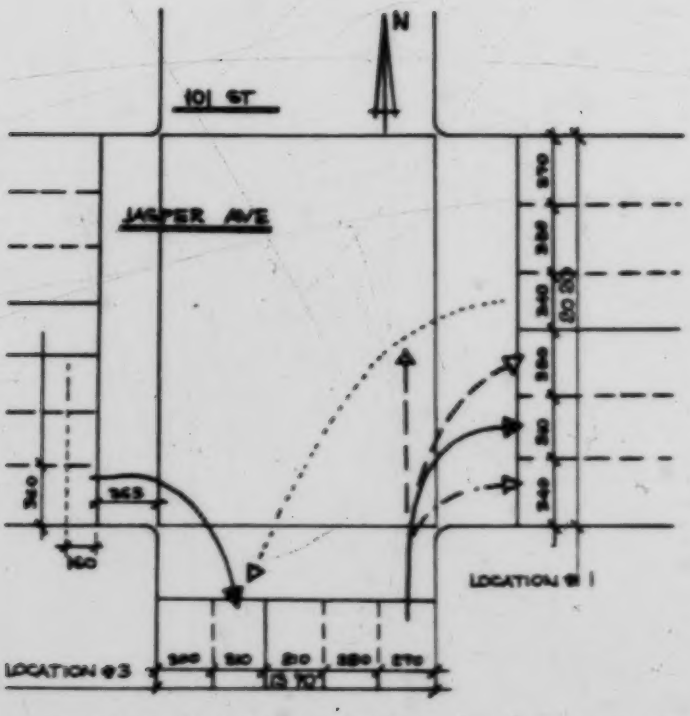


Figure 4.1 Location #1 and location #3 (Jasper Ave. - 101 St. E-bound; S-bound)

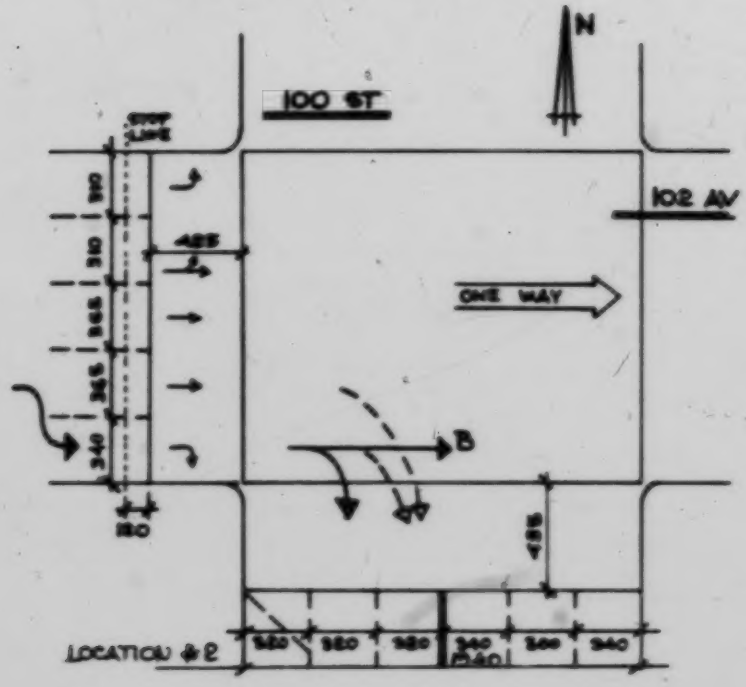


Figure 4.2 Location #2 (102 Ave. - 101 St. S-bound)

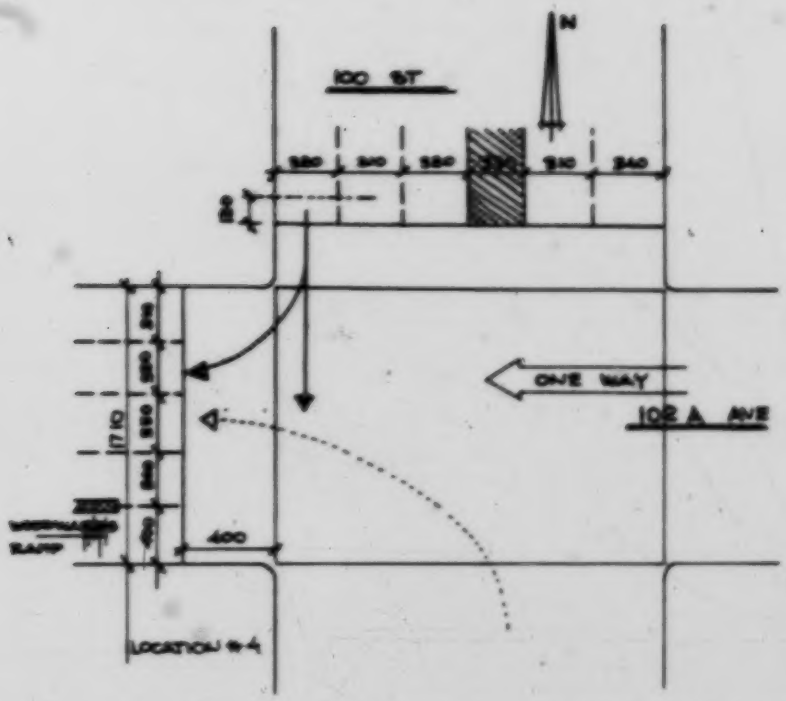


Figure 4.3 Location #4 (102A Ave. - 100 St. W-bound)

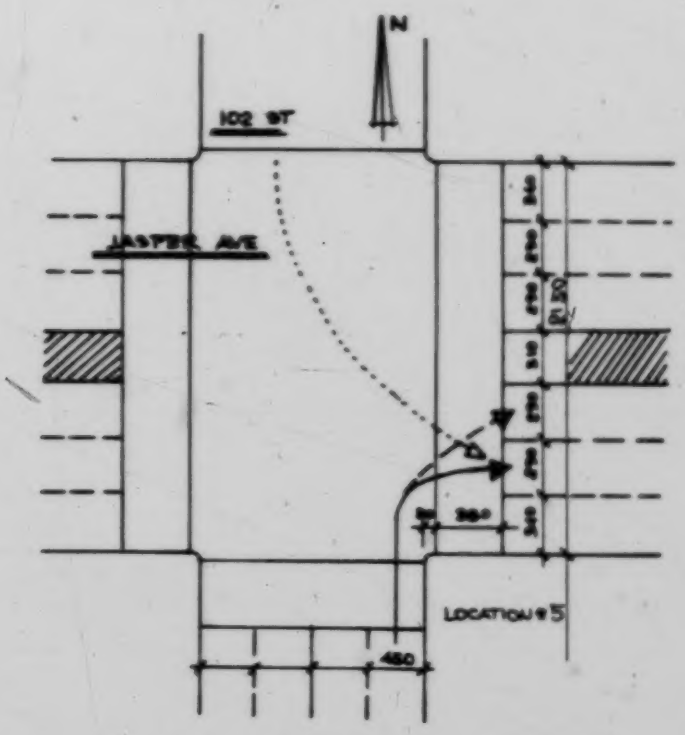


Figure 4.4 Location #5 (Jasper Ave. - 102 St. E-bound)

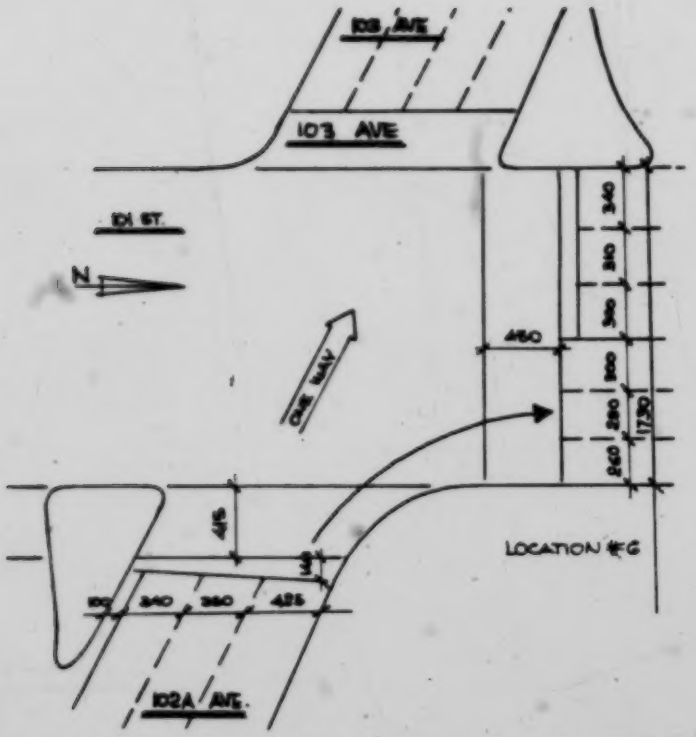


Figure 4.5 Location #6 (102A Ave - 101 St. N-bound)

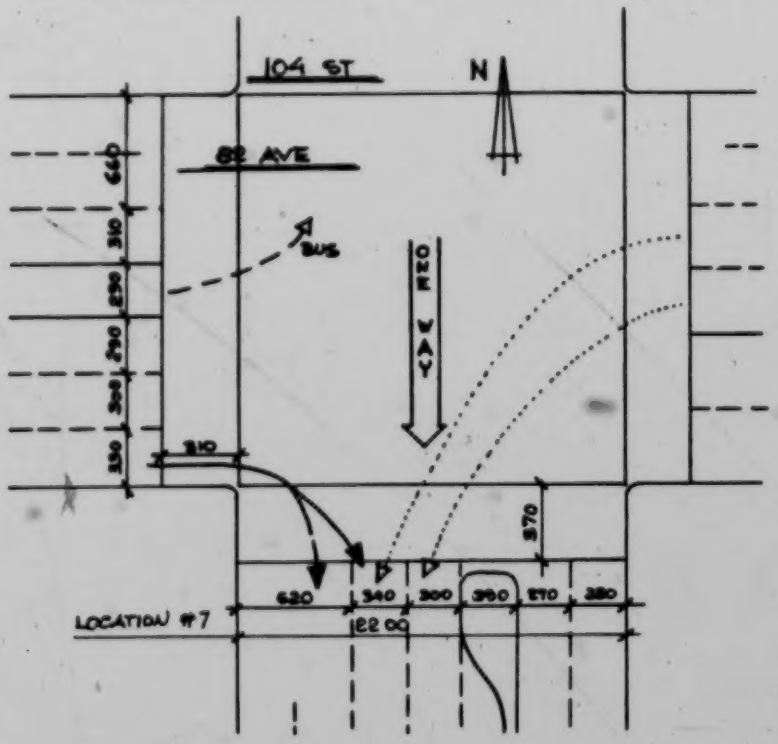


Figure 4.6 Location #7 (82 Ave. - 109 St. S-bound)

high loads at certain intersections. This effect helped to evaluate extreme conditions which are commonly absent.

The lanes with public transit stops and shared lanes were also observed. The behaviour of drivers in these conditions was generalized.

4.3. Data gathering

The data for this research were collected during the summer of 1983 and some additional measurements were collected in the summer of 1984.

4.3.1 Arrangement at the observation point

Clear visibility of all observed factors during the data collection was the prime consideration. Therefore the observation point was chosen as close to the observed zone as possible. Figure 4.7 shows the arrangement at the observation spot.

4.3.2 Method of data counting

The procedure adopted for data collecting was slightly different from the method used for straight through saturation flow evaluation (as presented in Ref. 8).

Pedestrian flow from nearside (P_n) was recorded when the pedestrians stop into the interaction zone (1). The number (N_n) was recorded when they left the nearside curb. Pedestrian flow (P_f) was recorded when pedestrians step into the interaction zone, in the moment when they pass the

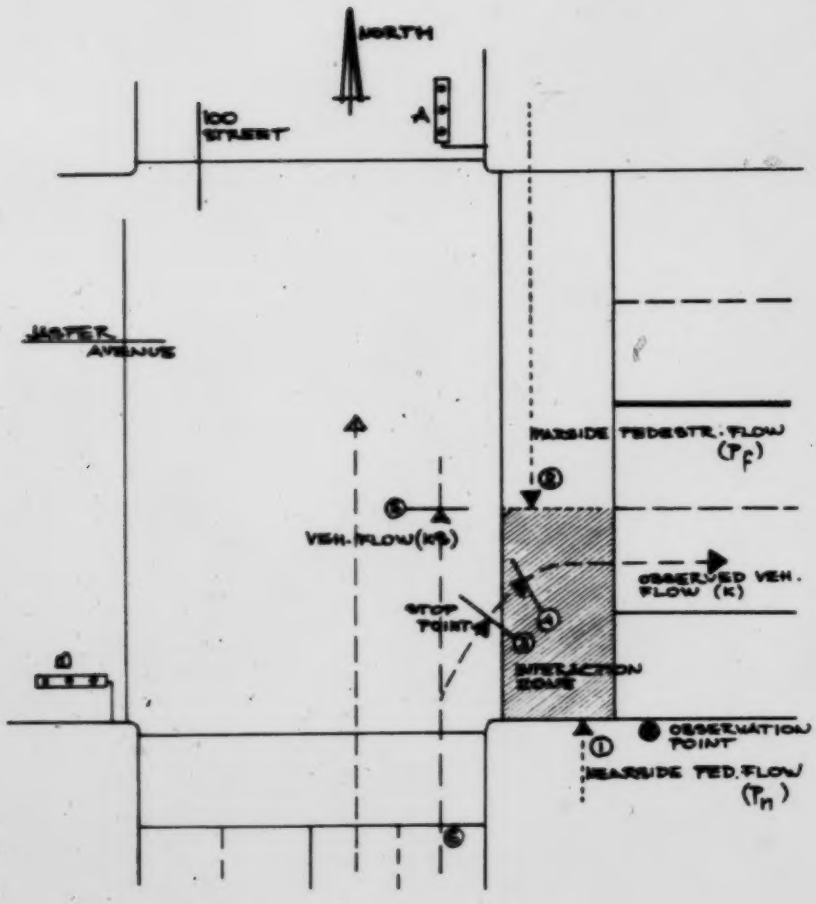


Figure 4.7 Situation at the observed location

farside border of the zone (2), which is the line between two traffic lanes.

Right turn vehicle flow (K) was recorded when the vehicle entered the interaction zone. Entering into the zone was considered when the vehicle moved across an imaginary line 1/3 of the width of the crosswalk (4). This action indicated that the vehicle would go through the crosswalk area and not stop.

Straight through flow (KS) was recorded when the vehicle passed the farside border of the intersection zone (5).

The original method of counting the vehicles for saturation flow evaluation counted the vehicles when the front bumper passed the crosswalk line (6) or beside the stopline. This method cannot be used in our case, because the right turning vehicle, already past point (6), is forced to stop at point (3) by the pedestrian flow. In that case the vehicle cannot be classified as "gone". The following vehicle going through passes line (6) also but it is stopped by the first vehicle and cannot be classified "gone" either.

The method modified for counting of right turning flow provides a slightly shifted saturation flow diagram (Fig. 4.8).

The shift at the beginning was caused by the greater distance through which the vehicle has travelled. This shift was eliminated at the end of the phase.. The end shift was due to the behaviour of the drivers. The drivers

turning right utilized the intergreen period more than in the straight through case. In many cases they utilized even the start of red. Call this period an "extended intergreen".

The beginning of the green, when the pedestrians are present was also correctly expressed. The vehicles counted on the stopline (6) or crosswalk line included headways. The first vehicle generally did not pass the interaction zone before the first pedestrian. Hence the vehicle creeping from stopline (6) to stop point (3) reduced its abnormal headway (Figure 4.9).

The modified method was used without any coversion and the results were directly compared with the previous research done on straight through saturation flow.

4.3.3 Requirements for manpower and tools

High pedestrian and vehicle flows made observations difficult for one person to carry out all data sampling. Therefore for the purpose of collecting data, two individuals and two tape recorders were required. One person recorded the number of vehicles going straight through (KS) and nearside pedestrian flow (P_n). The second person recorded farside pedestrians (P_f) and the right turning vehicle flow (K).

Measurement of cycle time, green time, amber, all red and pedestrian walk time required stopwatches. Each person used a stopwatch for continuous control of time slices. Geometry of the location was measured with a tape measure.

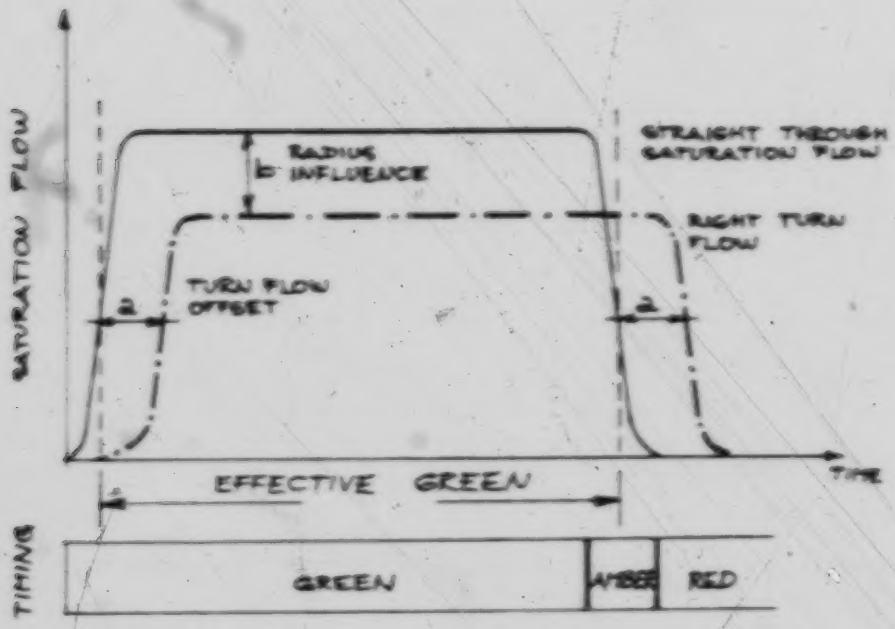


Figure 4.8 Comparison of straight through and right turn saturation flow (for zero pedestrian flow)

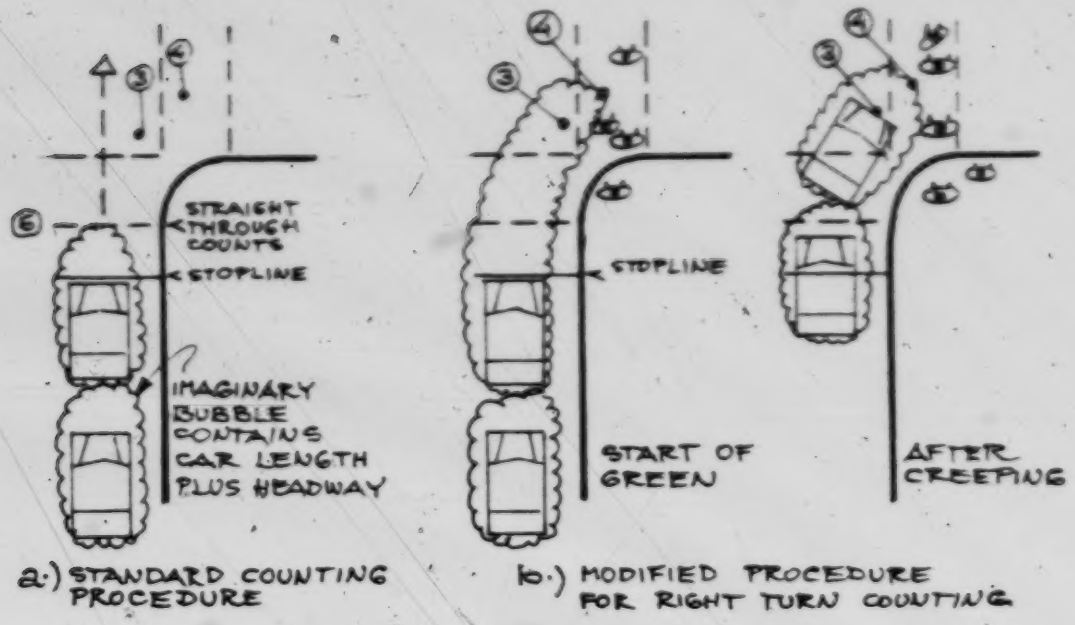


Figure 4.9 Example of direct and modified counting methods

4.3.4 Procedure

Data collection at all locations followed the standard procedure as described below.

A. Geometrical parameters of the selected location were measured and a field sketch was drawn. The sketch contained width of streets, width of crosswalks, width of lanes, offset of crosswalks from the corner, length of interaction zone and turning radius. The sketch also showed direction of turning movement, names of streets and the geographical orientation.

B. The field diary contained initial information such as: date, day of the week, starting and ending time of measurement, weather conditions, conditions of the pavement, names of observers, and additional observations (i.e. obstrutions, traffic pressure, etc.)

C. Time checking was provided before any observation started (proper recorder speed and function). Marking of 10 second slices was set prior to the regular counting. Then the irregularities, if any, were found and adjusted easily. The same recorder was used later in the office to avoid possible speed deviation. Again, the 10 second timing test was controlled prior to data presentation.

D. Timing conditions were recorded for each set of data. They were: cycle time, green time, amber, red, all red and pedestrian walk.

E. Pedestrian flow volumes were recorded for each 10 sec "slice" as well as volumes of vehicular flow. The

passenger car unit concept was being used in this research. Vehicles were recorded according to their categories as: car, truck, big truck, semitrailer, bus, motorcycle and specials.

F. Every change of situation was recorded (i.e. obstruction of any lane, interrupted vehicle flow, timing changes etc.)

G. Procedure of data collection was carried out in a simple format:

a. when the signal light for the street with interaction zone turned amber recorder was turned on and recorded "ready".

b. when the light (on the other console) for pedestrians turned green, "go" was recorded.

c. when the pedestrians stop into the interaction zone, "one, two, three etc." was recorded dependent on the group magnitude.

d. when the vehicle passed 1/3 of the interaction zone and moved forward, "car, truck, bus etc." was recorded.

e. when the signal for pedestrians turned red "stop" was recorded.

f. the counting of vehicles and pedestrians continued.

g. end of cycle was recorded as "amber, red" when the particular signal started flashing.

Procession of "green, stop, amber, red" in each cycle

or so called "time markings" allowed continuous time checking later in the office.

4.3.5 Data manipulation in the office

All the recorded data was transferred to a simple format data sheet. These sheets were then translated and stored in computer data files.

The tape recorders were first checked with stopwatches. Any differences between recorded timing and time imdicatd by the stopwatch were taken into account. According to the differences, the data were properly adjusted. Significant timing disproportion indicated possible battery trouble, hence the data was discarded.

Following the timing check, by using the stopwatch, the data from each cycle was transferred from the tape into the "format". An example of this format is given in Table 4-2. An attempt was made to distinguish between the first and the second half of each 10 second slice. It contributed to the evaluation of drivers and pedestrians behaviour.

Information or data stored in "format" were translated according to the pcu concept (Ref. Chapter 2) and restored in so-called "rough" computer files for the next manipulation.

4.4. Observed characteristics

4.4.1 Locations

The typical location found in Churchill Square was also

C	V	0 - 10s	- 20s	- 30s	- 40s	- 50s	COMMENTS
1	TH	112	3m1 2	11 1			OBSTRUCTION IN DISCHARGE LANE 20 M FROM CROSSWALK
	TF		252	1			
	K	C	C	B T	C C		
	KS			C			
2	TH						
	TF						
	K						
	KS						

Table 4-2 Example of "format" table

observed in the summer of 1982. The Edmonton method was based on these data. Seven sets of data with three different timing conditions were obtained at this location in the summer of 1983 and compared with the data obtained in 1982. For geometrical conditions refer to Figure 4.2.

Other turning conditions were observed at intersections presented in Figs. 4.1 and 4.3. These conditions were classified as similar and comparable with the first location according to the geometry and timing.

4.4.2 Timing and geometry

Different timing or geometry are presented for locations in Figures 4.4 to 4.6. The data from these locations allowed the evaluation of the influence of different timing and geometry on right turn vehicle flow.

4.4.3 Interaction zones

The interaction zones were found, in most cases, to be wider (actually longer) than those used by Yagar (Ref. 10) for his model, because vehicles utilized the second lane for discharge even when the first lane was unobstructed. For this reason a longer interaction zone was considered for calculation.

4.4.4 Traffic characteristics

The utilization of the intersection zone depends not only on geometric and timing factors but also on traffic

conditions. It appears to be important to include so called traffic pressure into evaluation.

Traffic pressure can be characterized as an emotional factor which changes the behaviour of the driver due to his feeling for responsibility to others (example: blocking the lane or stop point) or to himself (example: being caught in congestion). Traffic pressure has negative effects (such as increase of accident rate as a result of overestimation of real abilities due to drivers nervousness; rudeness and disobedience of drivers etc.) and in some cases also positive effects (increase of basic saturation flow and consequently the increase of capacity; increase of driver's ability to solve given situation).

The characteristics observed during the research are described in Appendix A.

4.4.5 Traffic fluctuation

Slight differences were detected between different observations of the locations due to the loading volumes. The most typical pedestrian and vehicle load conditions were found between 11:30-13:30 and 15:30-17:30 in the downtown areas. At the location on 82nd Avenue, high peaks occurred between 12:30-13:30 and 16:00-17:00.

An adequate pedestrian flow was not present in the morning. For this reason morning peaks have not been considered in this research. In the later parts of the afternoon peak hours oversaturation and congestion on the

streets made counting impossible. Streets were blocked and no rules could have been applied.

The University Games 1983 significantly influenced the conditions in the downtown area. Higher pedestrian flows interacted with higher vehicle flows during the daytime especially at lunch time. This event presented a great opportunity to observe extreme situations.

4.4.6 Special conditions

The "standard" conditions were characterized as:

A. unobstructed approach (exclusive right turn) lane
B. unobstructed discharge lanes
C. standard timing
D. interference with pedestrians

Other conditions which are not characterized as "standard" are called "special" ones. They are:

A. congested situation at the intersection
B. undersaturated condition
C. obstructions in discharge lanes
D. obstruction in approach lanes
E. lanes with right turning and straight through traffic

The special conditions in majority of cases do not cover the topic of this research. The cycles with special conditions were excluded from evaluation.

Because of importance of these conditions they were categorized and described in Appendix A.

4.4.7 Vehicle performance

The pcu concept was used for evaluation of the right turn problem. Comparative units pcu (Passenger Car Units) were used for all three models. Saturation flow was also expressed in pcu per hour of green. The Canadian Capacity Guide (Ref. 8) uses the same comparable units for all expression of flow. Therefore the vehicles with different levels of performance were converted in the following manner:

a. van, pickup, passenger car = 1.0 pcu
b. single unit trailer = 1.5 pcu
c. truck = 1.5 pcu
d. multiunit trailer = 2.5 pcu
e. multiunit trailer (loaded) = 3.5 pcu
f. bus = 1.75 pcu
g. motorcycle = 0.5 pcu

The Capacity Guide matches the average equivalent for vehicles with dual tires to the Toronto report, however the detailed conversion differed slightly.

4.4.8 Additional observations

Yagar's method operated with the input data which were directly measured. They were: cycle and green time, street width and interaction zone length. The additional data such as group size, pedestrian speed, gap size, probability of acceptance etc. were already verified at Waterloo. The same

values were used for comparison with other methods during this research. These data were also roughly checked in Edmonton by observing a few cycles.

A. The average speed of pedestrians was confirmed to be 1.2 m/sec. The middle sized groups of pedestrians were moving faster than 1.2 m/sec. The speed of bigger groups was lower due to the friction among pedestrians. The speed of small groups or randomly distributed pedestrians was also found to be smaller due to the lack of traffic pressure. The fluctuation of pedestrian speed was so significant that the average of 1.2 m/sec was used for this research in accordance with Waterloo findings.

B. The group size also varied significantly. The most common were found the groups with size of 2-3 pedestrians. In the majority of cases, the random flow consisted of one pedestrian per group. Groups consisting of twelve or more pedestrians were also found. The average size of 1.7 pedestrians/group was used for calculations.

C. The most difficult part of the observation was the determination of the gap between pedestrians acceptable by penetrating vehicles. A significant variation was found (Table 4-3). For the calculation data were used as presented by Yagar.

D. "Traffic pressure" played an important role on drivers' and pedestrians' behaviour. This factor was observed during the University Games in the summer of 1983. The highly loaded intersections handled extreme

NUMBER OF PASSING VEHICLES	GAP SIZE (sec)	
	BY YAGAR	OBSERVED
1	5	4-7
2	8.6	7-9
3	12.2	9-12
4	15.8	10-15

Table 4-3 Gap size accepted by penetrating vehicles

traffic well, as pedestrians and drivers reacted sharply as compared to the situation expressed as a "lazy Sunday afternoon".

E. Cycles with any abnormalities were discarded from this study. The research provided an evaluation of standard conditions. The knowledge gained during these observations was used for modification of mathematical results.

CHAPTER 5 - COMPARISON OF THE THREE METHODS

5.1. Computer files

The extensive amount of data required computer processing. The collected data were transferred from "format" sheets to the computer files. These files had the same arrangement and type of data information as formats (numbers of vehicles converted into pcu as well as numbers of pedestrians entering the intersection zone from both sides).

The comparison of the three methods required different data inputs for each of them. For this reason the data were manipulated and formatted to be suitable for each type of analysis. Numbers of vehicles and pedestrians were converted into flows per hour of green.

Analysis used for development of Edmonton method also required the input to be properly formatted. Combination of data and further adjustments dictated another data files manipulation.

Basic files (rough, manipulation, working, Yagar's, combined) are described in Appendix B. Intermediate files created for modification of inputs were used temporarily and later destroyed.

5.2. Programs

For the data conversion and creation of different types of files had to be written manipulation programs such as:

TRAN - Program transferred the rough data files into manipulation files. Numbers of units were transferred into flows.

REG - Program formatted data input for regression routine POLYREG. Pedestrian and vehicle flows were presented for time "slices" on cumulative basis in working files.

AVR - Program averaged the data for evaluation on longer time basis.

POIN - Program selected the required data from the files for specific purposes.

CAL - Program expressed the regression formula in coordinates for graphical expression.

For the evaluation of collected data the programs for computation had to be written. Program COMP compared the three available methods with observed data from selected locations. This program also presented the results for graphical presentation.

The University software was used for regression analysis, graphical presentation of results, and for organizing of data files.

STPK - Program POLYREG was used for regression analysis of data. Polynomial function represented the basic trend.

GENPLOT - Regression equations were expressed graphically by using this program.

The programs are presented, described and listed in

Appendix B.

5.3. The methods of comparison

5.3.1 Program COMP

The right turning vehicle flow was evaluated by the three different methods previously described. Each method has its own advantages. The research objective was to find the most practical method. The computer program COMP was used to compare the three existing methods. The data obtained in seven locations were used for evaluation.

Program COMP used the percentage differences between measured and theoretical values for comparison and calculated them as mathematical average (i.e. + or - signs of the differences were recognized). The smallest mathematical difference indicated the best descriptive method. The principle of mathematical difference is explained in Appendix B (Program COMP).

5.3.2 Approximation

The evaluation using the program COMP was based on green interval as expressed by Yagar's method. The input file contained the data for each 10 second interval. Yagar's method used data according to the real green time. For locations #6, #7 and locations #2 and #3 with green interval 40 or 30 seconds the data matched exactly. For locations #1, #2, #3 and #5 the green interval varied as 31 sec, 33 sec and 39 sec. For location #4 the green time was

22 sec. In these cases the data from the "slice" 0-40 sec were used for locations #1, #2, #3, #5 and "slice" 0-30 sec was used for location #4. This procedure included approximation which would be significant if involved in calibration of Yagar's method. For the purpose of rough comparison this procedure was considered as acceptable.

5.3.3 Programs STPK (POLYREG) and GENPLOT

Statistical package STPK(POLYREG) was used to determine the fit of measured data with the Toronto equation.

Graphical plot (Program GENPLOT) of measured data and values calculated by the three methods under evaluation is presented in Figures 5.1 to 5.9. Figure 5.10 presents the results from location #6 in numerical form. Figure 5.11 presents the comparison with the Toronto method.

5.4. The results of the comparison

5.4.1 Toronto method

It was found that the computed results from the Toronto method were higher than the observed values (Fig. 5.1 to 5.9). The mathematical difference between the evaluated samples was in the range +50% to -70% depending on location. This was not surprising, since it has been recognized that driver and pedestrian behaviour is different in both cities.

Regression of the first degree was used (in location #2) for all time "slices". The obtained regression

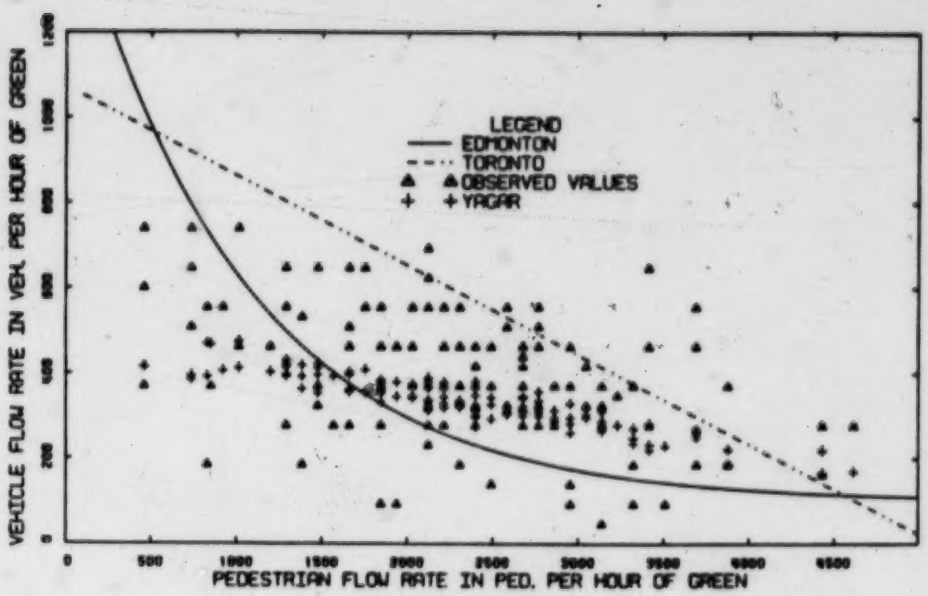


Figure 5.1 Location #1 - Jasper Ave. - 101 St. East bound

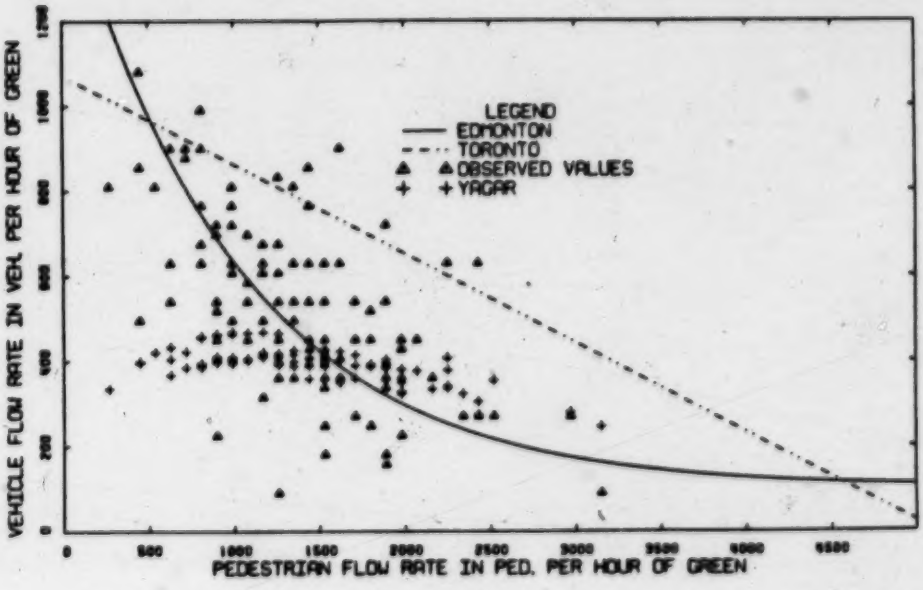


Figure 5.2 Location #2 - 102 Ave. - 100 St. South bound

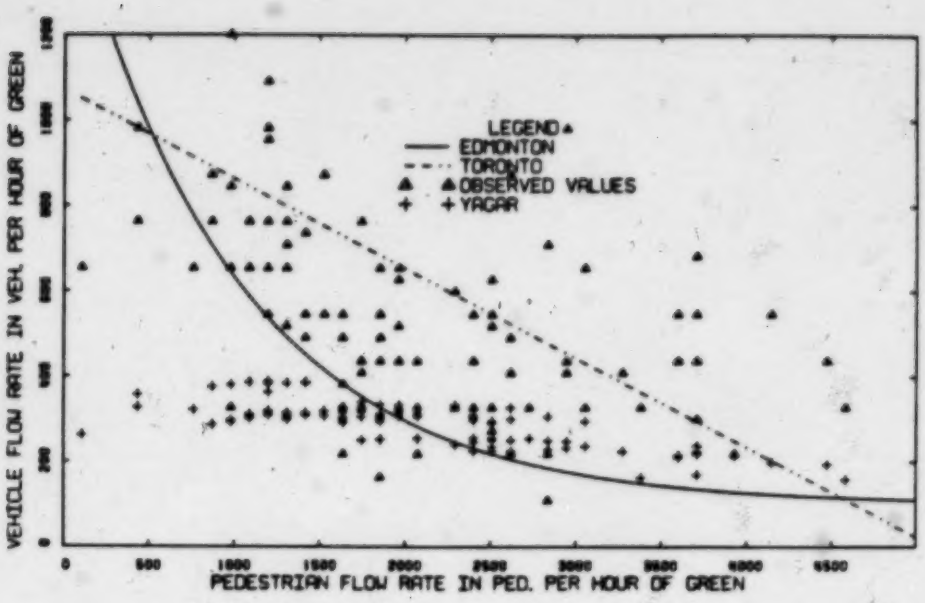


Figure 5.3 Location #2 - 102 Ave. - 100 St. South bound

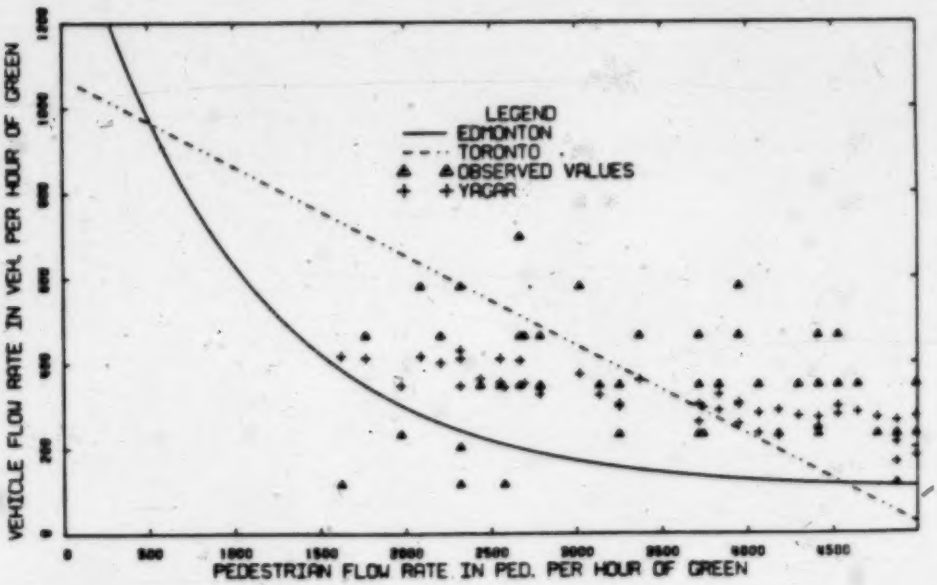


Figure 5.4 Location #3 - Jasper Ave. - 101 St. South bound

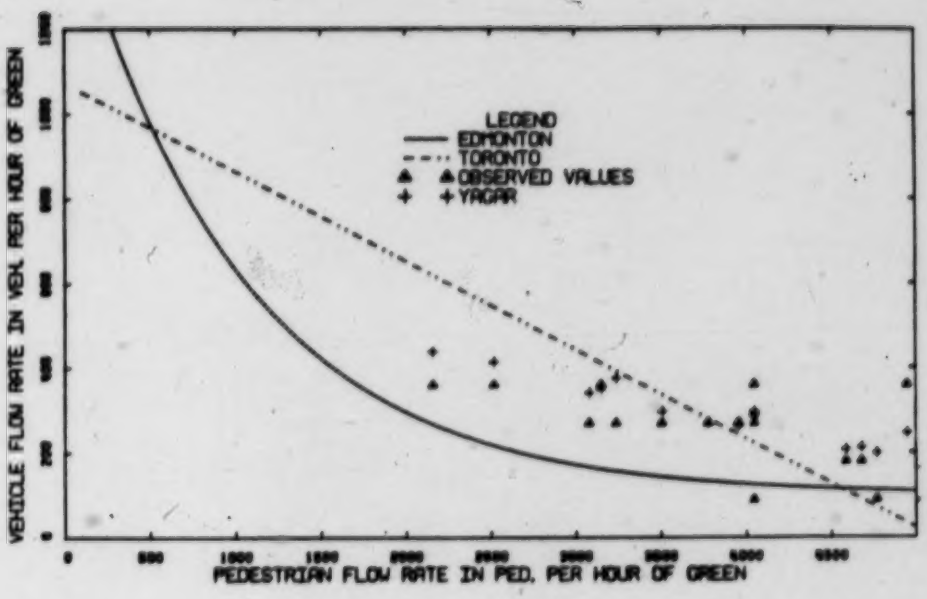


Figure 5.5 Location #3 - Jasper Ave. - 101 St. South bound

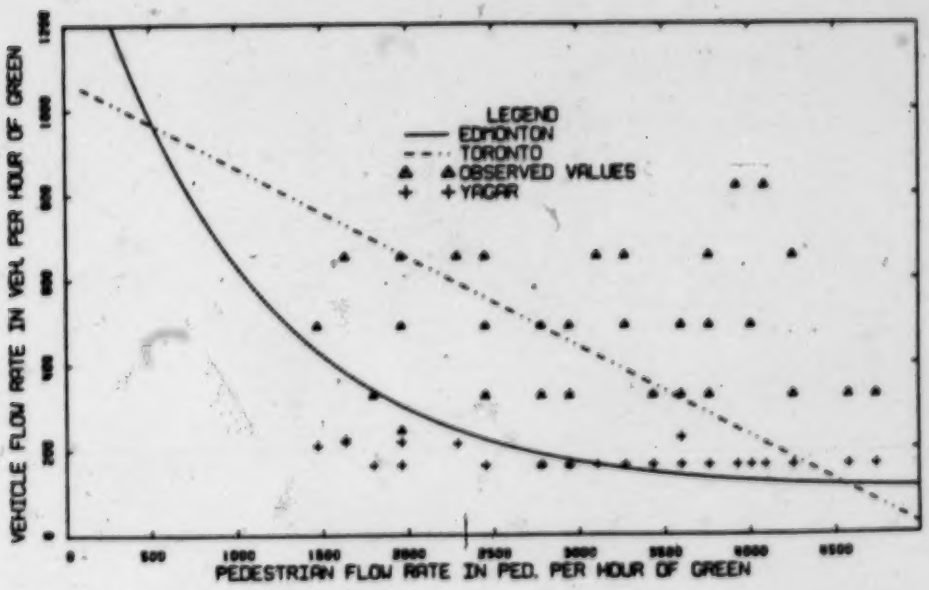


Figure 5.6 Location #4 - 102A Ave. - 100 St. West bound

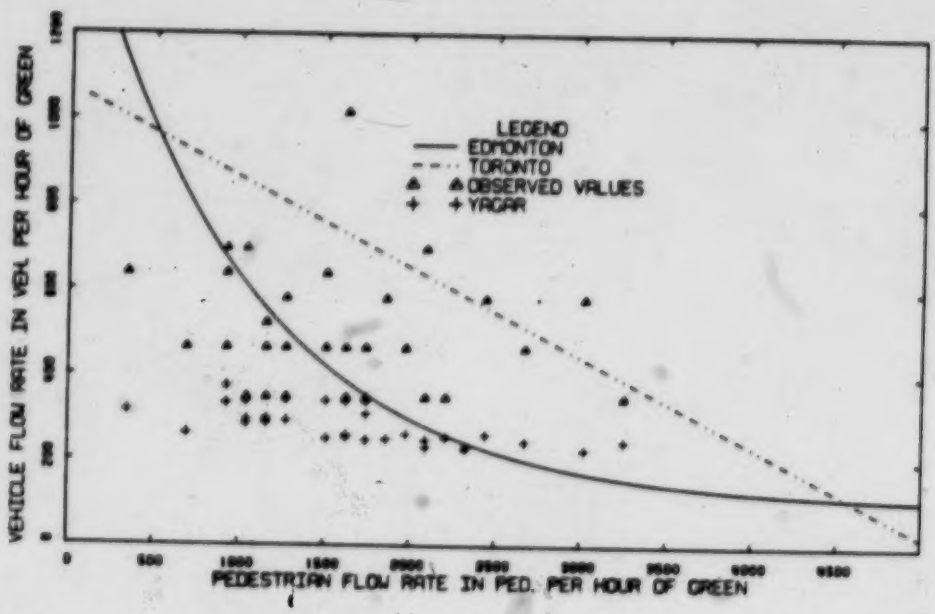


Figure 5.7 Location #5 - Jasper Ave. - 102 St. East bound

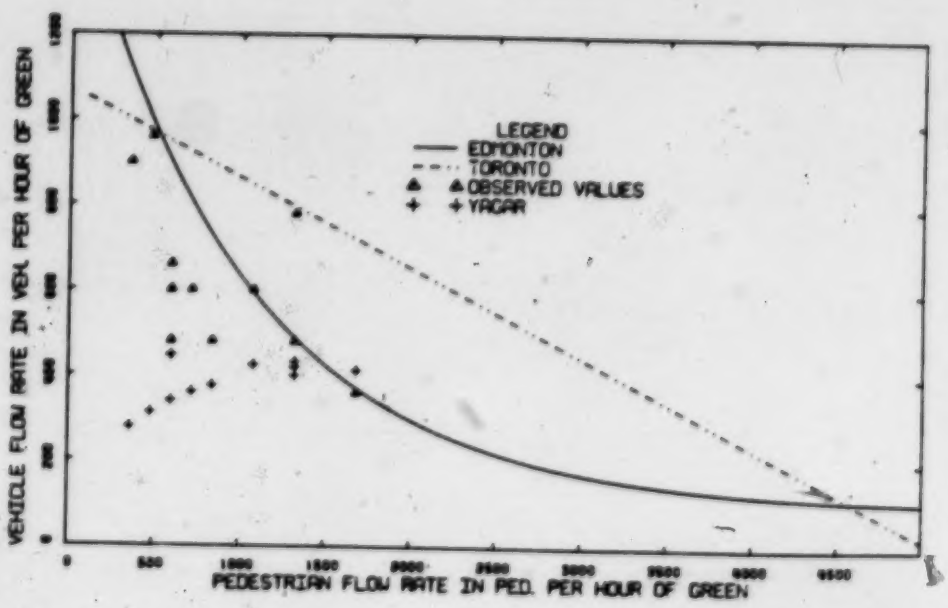


Figure 5.8 Location #6 - 102A Ave. - 101 St. North bound

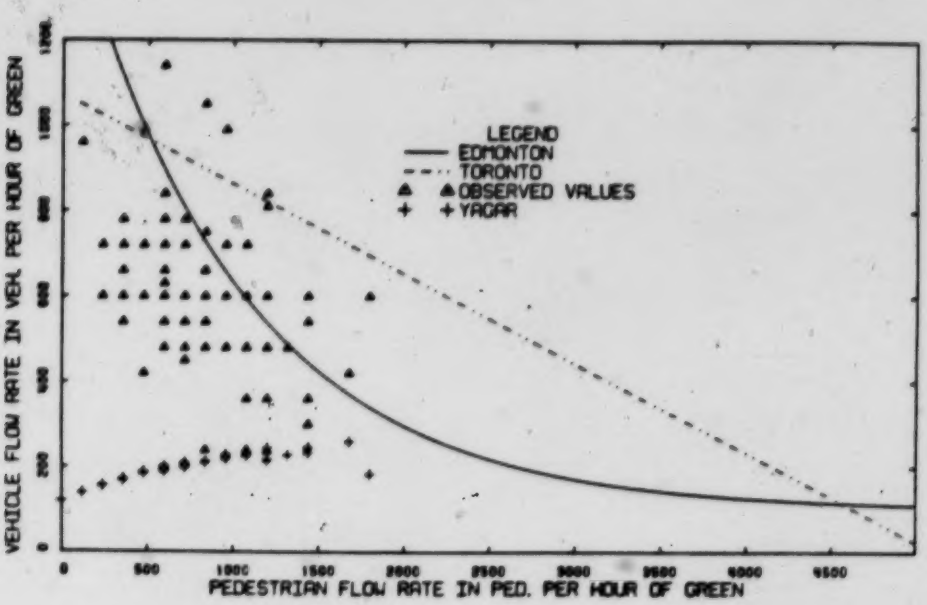


Figure 5.9 Location #7 - 82 Ave. - 109 St. South bound

	PED/G	REAL V/G	TOR V/G	ED V/G	YAG V/G	YAG V/C	INIT	REST	TOR%	ED%	YAG%
2	1080.0	600.0	846.2	892.4	434.3	3.54	0.0	3.54	41.	-1.	-28.
3	720.0	600.0	921.8	908.8	360.6	3.01	0.0	3.01	54.	34.	-40.
4	1320.0	480.0	795.8	497.3	418.5	3.50	0.0	3.50	66.	2.	-13.
5	360.0	800.0	987.4	1111.6	278.5	2.33	0.0	2.33	11.	24.	-48.
6	1320.0	780.0	795.8	497.3	432.2	3.60	0.0	3.60	2.	-36.	-45.
7	1320.0	480.0	795.8	497.3	399.8	3.33	0.0	3.33	66.	2.	-17.
8	600.0	660.0	947.0	895.8	445.7	3.72	0.86	2.76	43.	36.	-32.
9	840.0	480.0	896.6	726.0	376.2	3.14	0.0	3.14	87.	51.	-22.
10	480.0	860.0	973.2	907.2	312.7	2.61	0.0	2.61	1.	4.	-67.
11	1680.0	360.0	730.3	370.3	412.3	3.44	0.0	3.44	100.	3.	15.
12	600.0	480.0	947.0	895.8	445.7	3.72	0.86	2.76	97.	87.	-7.
13	600.0	600.0	947.0	895.8	338.7	2.83	0.0	2.83	58.	49.	-43.
14	600.0	600.0	947.0	895.8	445.8	3.72	1.00	2.72	58.	49.	-26.
15											
16	AVERAGE REAL		AVER.TOR.		AVER.YAG.	DIF.TOR.		DIF.EDM.	DIF.YAG		
17	613.84		886.91		393.00	44.		21.	-36.		

Figure 5.10 Output example from program COMP - Location #6

equations are presented in Table 5-1 and graphically in Fig. 5.11. It can be seen that the trend of relationship between pedestrian and vehicle flows is almost identical with Toronto line (the slope). The evaluated data were available in the middle region (500-3000 pedestrians/h green). The total difference in this region was found to be identical with difference in saturation flow values for both cities (Fig. 2.2). The ends of the regression lines however did not express the observed reality.

5.4.2 Edmonton method

The preliminary exponential form of the Edmonton regression function was available from a pilot study 1981-82. This exponential curve was superimposed on measured data.

As can be seen in the Figures 5.1, 5.2 and 5.7 the curve represents a reasonable average trend within the measured values. Figures 5.3, 5.4, 5.5 and 5.6 indicate that the mean values were higher. Figures 5.8 and 5.9 show that the measured values are lower than the computed ones. The mathematical difference between the evaluated samples was in range +25% to -58%. Overall the curve seems to represent the reasonable balance, especially for low pedestrian flow. Further refinement of the Edmonton function is presented in the next chapter.

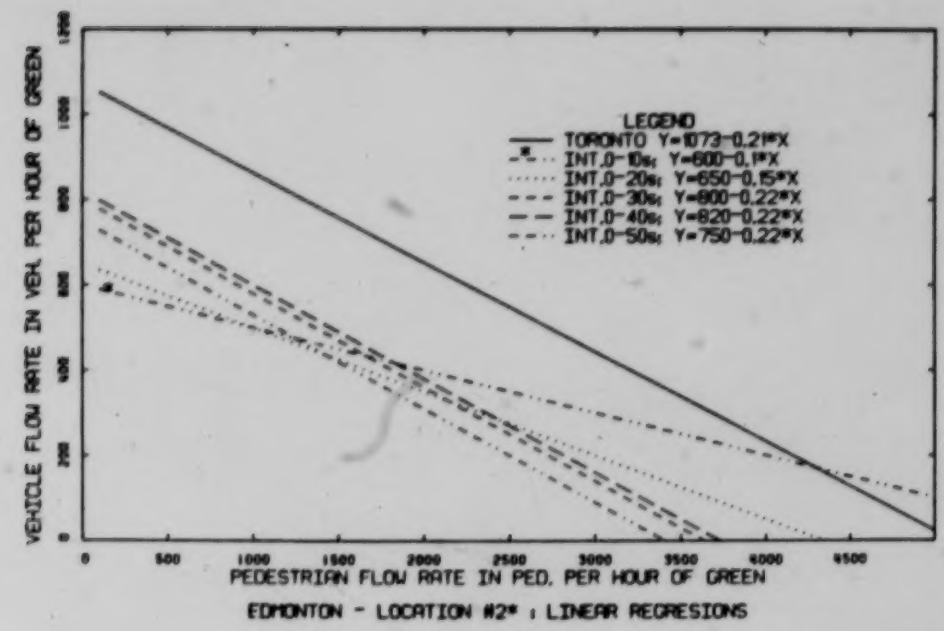


Figure 5.11 Comparison of linear regressions with Toronto method

TIME SLICE	REGRESSIONS FROM OBSERVED DATA	TORONTO FORMULA
0-10	Y= 600 - 0.1 x	Y= 1073-0.21x
0-20	Y= 650 - 0.15x	
0-30	Y= 800 - 0.22 x	
0-40	Y= 820 - 0.22x	
0-50	Y= 750 - 0.22x	

Table 5-1 Comparison of linear regressions with Toronto method

5.4.3 Yagar's method

As can be seen in Figures 5.1 to 5.5 for pedestrian flow region of 1500 and more ped/h green matches the measured data well with the calculated ones. Figures 5.6 and 5.7 indicate values somewhat lower than those of measured, but a correct trend. Major problem may be found in Figures 5.2, 5.3, 5.8 and 5.9. The illogical trend appears in region 0-1300 ped/h green.

While in reality for low pedestrian flow some vehicles could complete their right turns, this calculation method indicated a zero vehicle flow during initial periods (Fig. 5.10).

The mathematical difference between the evaluated samples was in range +67% to -60%. The method could be used for the range 1500-5000 ped/h green but not for lower pedestrian flows.

5.5. Conclusion

Comparison of the three methods with measured data from seven Edmonton locations proved that all three methods provide reasonably good approximation of the right turning problem. They all worked well in the range of pedestrian flow 1500-3500 ped/h green. Differences among these methods can be explained by different environmental, geometric and traffic conditions involved.

Problems were found in the regions with small (0-1500 ped/h green) and high (4000 and more ped/h green) pedestrian

flows. Neither Toronto nor Yagar's method covered these regions satisfactorily. The Edmonton method provided promising results in all regions with caution that further development is needed. Difference among the results from different locations introduces the necessity of adjustment factors for geometry and timing to be taken into account. Solution of this problem is presented in the following chapters.

CHAPTER 6 - FURTHER DEVELOPMENT OF EDMONTON METHOD

6.1. Programs used for development

For the development of the Edmonton method several programs were written and also the routines were used. They were:

A. STPK - Statistical package includes two used routines. Program POLYREG was used for regression analysis of data. Polynomial function represented the basic data trend. Program HIST was used for formation of normal distribution curves according to the observed data.

B. GENPLOT - This routine was used for graphical expression of results.

C. SORTIT - Routine was used for sorting the data in selected order for later evaluation.

D. FIN - Program adjusted the observed data according to the traffic and geometric conditions.

E. HIS - Program calculated averages of pedestrian and vehicle flows for each 10 second "slice". Histogram based on these averages was used for selection of highest saturation flow as well as for observation of interaction during the cycle time.

The programs for manipulation with files were also used. Appendix B presents the programs for further reference.

6.2. Method used for development

The observations made at locations in Edmonton pointed out that the relationship between pedestrian and vehicle flows was nonlinear. Examination of the data from seven Edmonton locations confirmed that neither Yagar's nor the Toronto method yield satisfactory results. The difference between the observed and the calculated values was significant. The Edmonton method was considered to be best suitable for further development since the analysis of observed data indicated a nonlinear trend.

6.3. Typical location

The first part of the evaluation was carried out on the samples from one "typical" intersection. Location #2 at Churchill Square (Figure 3.5) yielded moderate and higher pedestrian flows as well as moderate or higher vehicle flows in the approach lane. The vehicles used unobstructed exclusive lane for right turns and unobstructed discharge lane. There was no interference of any sort. The straight through traffic flow was negligibly small. Timing conditions were: cycle time 80 or 90 seconds, green interval 37 or 48 seconds, pedestrian walk 18 or 25 seconds and amber interval 3 seconds.

The data collected in 1982 were available for this intersection. Information regarding timing conditions, traffic conditions, counting method etc. was not available

for this set of data. This pilot research was taken as supporting observations and was compared with the final result (curves) graphically.

Later data from all the seven locations were used for determination of additional factors influencing the right turning vehicle flows.

6.4. Description of procedure

6.4.1 Least square method

Statistical package (STPK) was available and its part (POLYREG) was used for regression analysis of data (Ref. 21). The program used least square method for definition of polynomial regression lines.

The advantage of polynomial line was that it provided a good trend in the region (1000-3500 ped/h green). Polynomial expression was used for improvement of definition function in the first step of evaluation to define the trend of relationship between pedestrian and vehicle flows (Fig. 6.1).

6.4.2 Data significance and sample size

From the statistical evidence (Refs. 18 to 20) a minumum of 30 observations for one set of data was determined as a sufficient sample. The samples with less data were later used for support when necessary.

The first task was to deal with the sensitivity of individual sets of data obtained from one location. It was

necessary to know whether or not each set differed significantly from the others. The regression analysis (using the STPK) were utilized for the evaluation of each data set. Each 10 second interval (or slice) was evaluated separately to provide a detailed picture regarding flows in the interaction zone. Regression analysis showed that some samples have the regression coefficients (0.12 to 0.86) which were not satisfactory (Fig. 6.2).

The first part of data evaluation proved that the analysis of small samples did not produce reasonable consistency of results, as one exceptional data point in a small set brought about the misleading results. This effect was more pronounced in the case of second degree (parabolic) curves. High fluctuation of results was evident in the analysis of small samples. They were significant and yielded wide spread. A typical example of evaluation of small sets is presented in Figure 6.3. For more examples refer to Figures C.1 to C.5.

A sample size of 130-200 observations was found to be reasonably adequate and acceptable for polynomial regression analysis. In this case an exceptional data point did not have a misleading influence on the final results. In some cases the analysis of sample with 30 data points would be adequate.

6.4.3 The degree of polynomial regression

The regression analysis of the first up to the fourth

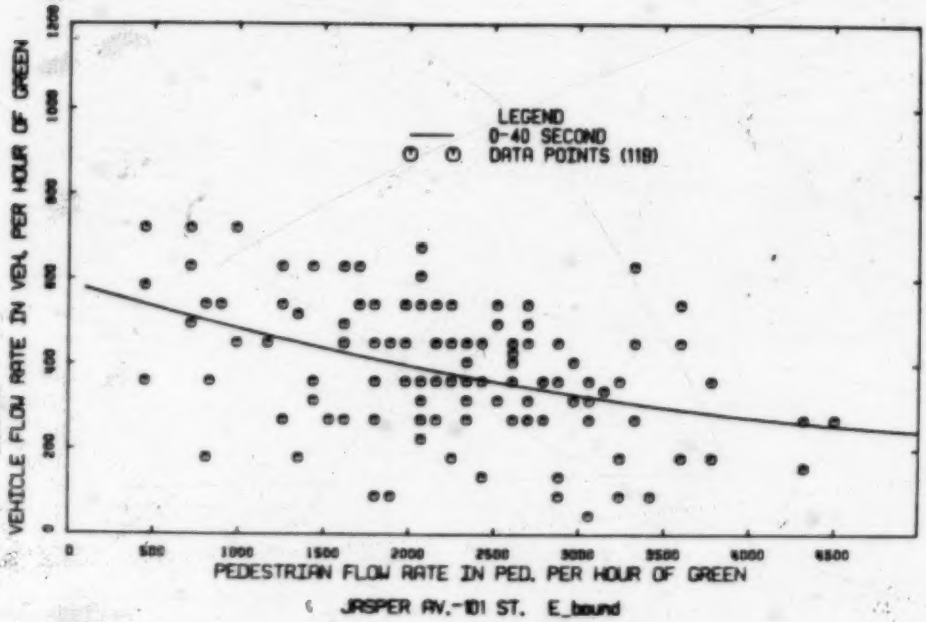

LEGEND
0-40 SECOND
DATA POINTS (118)
VEHICLE FLOW RATE IN VEH. PER HOUR OF GREEN
PEDESTRIAN FLOW RATE IN PED. PER HOUR OF GREEN
JASPER AV.-101 ST. E_bound

Figure 6.1 Example of polynomial regression and its graphical expression (GENPLOT)

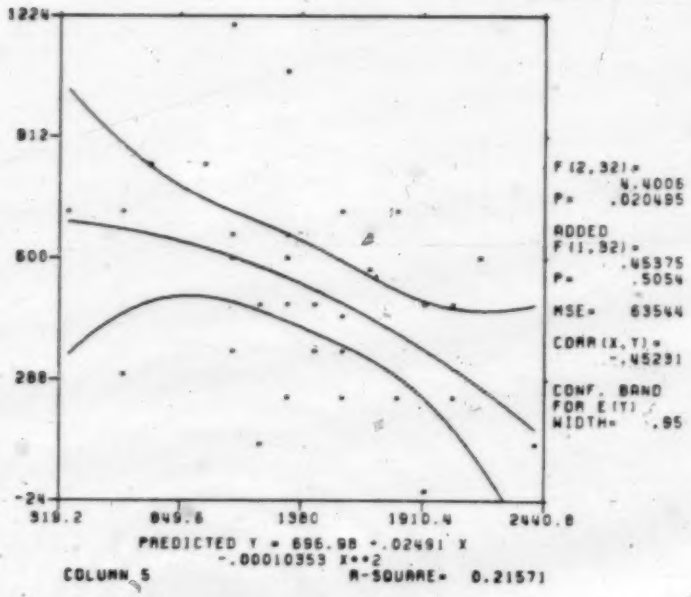


Figure 6.2 Example of polynomial regression (POLYREG) line together with regression coefficient

degree were applied to all data sets. The curves of third and fourth degree were not practical (Figures C.11 and C.12). According to previous experience the second degree curve was the best rough approximation and description of the examined trend. It was used for further development.

Figures C.1 to C.5 indicate that the middle region was described well but the ends of the curve needed adjustment to avoid the error contained in the statistical procedure.

This phenomenon was explained by using the regression procedure without any logical constraints. Unconstrained regression followed strictly the mathematical method in our case minimizing the squares and following the mathematical character of each degree. For this reason the ends of the curves went in illogical directions.

6.4.4 Averaged data

The attempt to arrange data sets in some other fashion was made to get sensible results. The data were smoothed or grouped and represented by one point. This procedure eliminated the influence of extremes and was found to be a good practice in some cases, especially for a large amount of data (Figure 6.4).

The smoothing was actually an averaging procedure. This method created great difficulty. Researched topic had its own specific conditions where time bias was involved. It was previously found that the measured saturation flow for individual cycles peaked more than the saturation flow

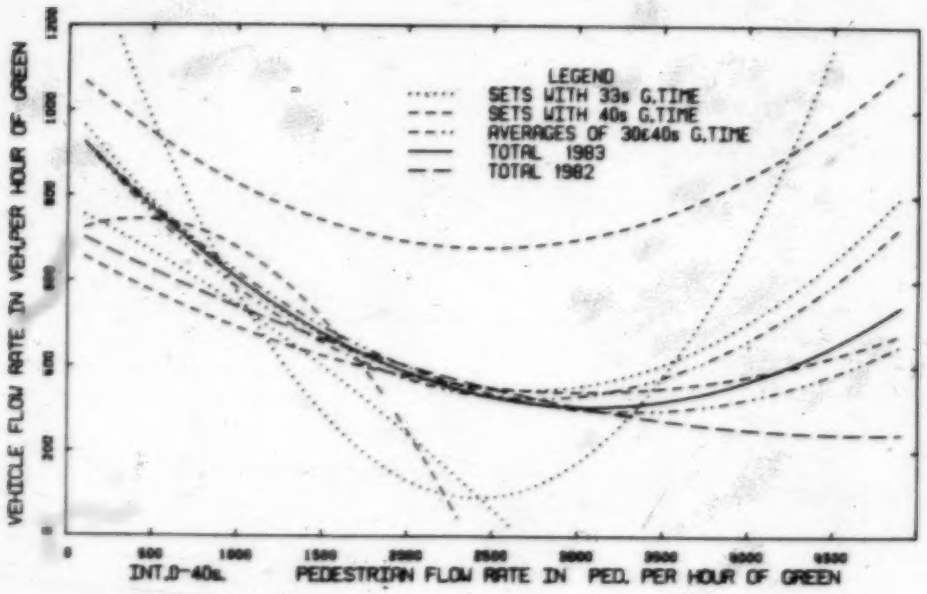


Figure 6.3 Regression analysis of individual data sets

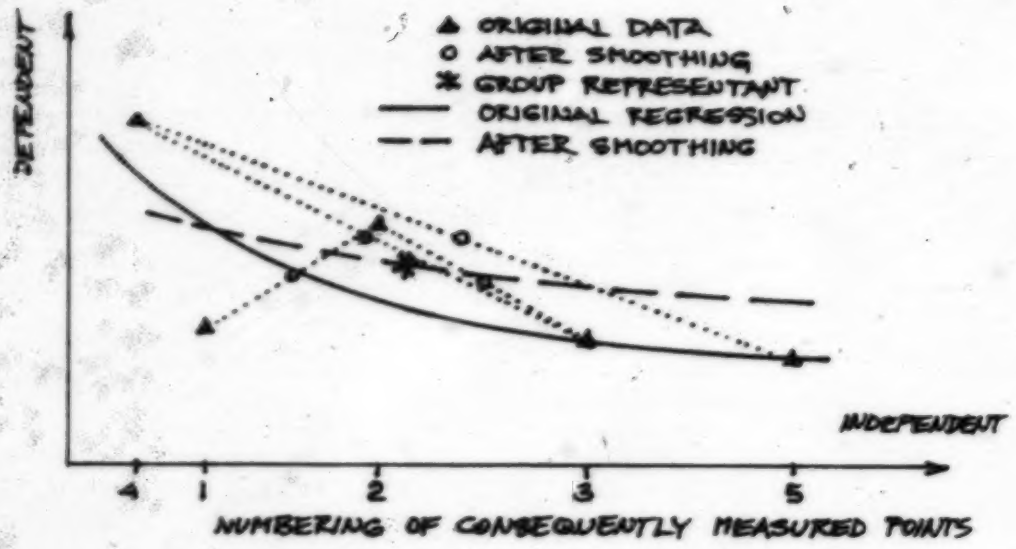


Figure 6.4 Effect of smoothing

for 5 minutes. The hourly measured flow had even much lower volumes. Thus the total magnitude of flow which is time biased depended on evaluated time base. The averaging of consequently measured cycles created the time biased value noncomparable with the pure flow.

Another problem was created by the fact that according to the pedestrian flow rate, the measured vehicle flows belonged to different categories. For example category 0-500 ped/h green could not be averaged together with the category of 5000-5500 ped/h green.

In the case, of consequently averaged cycles, smoothing produced wrong results. The average category (2500 ped/h green) never yielded predicted value, due to the nonlinear trend involved.

The previous sorting and then averaging of data in categories was found sensible only partially. Especially in the category of 0-500 ped/h green the averaging process destroyed the data sensitivity. Generally the bias of smoothing was noticed to be misleading in the situations where data for extremes were required. For the conditions with few measured values (pedestrian flow close to zero) all data points were desired, however they were lost during the smoothing procedure. In some cases the results appeared to be even illogical (Figures 6.5 and C.6 to C.10) but the trend indication can still be used.

For the above reason the smoothing process was used only for graphical expression of observed data. The

analysis was applied to the original measurements only.

6.4.5 Combined data

The regression analysis was applied to the combination of all sets of data from the same location and the same timing conditions. The geometric conditions remainded unchanged as the sensitivity to the timing was analyzed.

The results from combined data provided reasonable results and identified the trend (Fig. 6.6 to 6.10). The Toronto regression line is superimposed for better orientation.

6.4.6 Geometry and timing

All data sets from one location, regardless of timing were evaluated as one set to analyze the intersection as to how it performed under the same geometric conditions. It was found that different timing conditions did not change the results significantly. Different timing can be distinguished in Figures 6.6 to 6.10.

6.4.7 Time base

The 10 second intervals (on the cumulative basis) were analyzed separately. Separate evaluation was needed since the final values for a total green interval did not provide in some cases good indication for the real trend. The trend was hidden and could not be identified by using only the last time "slice" (0-50 sec), especially from samples with

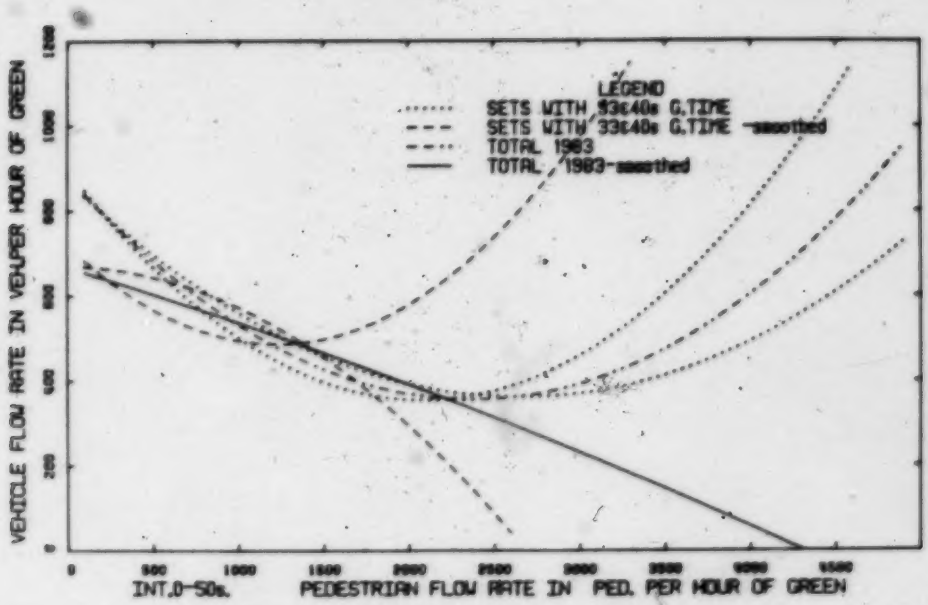

LEGEND
SETS WITH 33&40s G.TIME
SETS WITH 33&40s G.TIME -smoothed
TOTAL 1983
TOTAL 1983-smoothed
VEHICLE FLOW RATE IN VEH/PER HOUR OF GREEN
INT.0-50s.
PEDESTRIAN FLOW RATE IN PED. PER HOUR OF GREEN

Figure 6.5 Regression of smoothed data

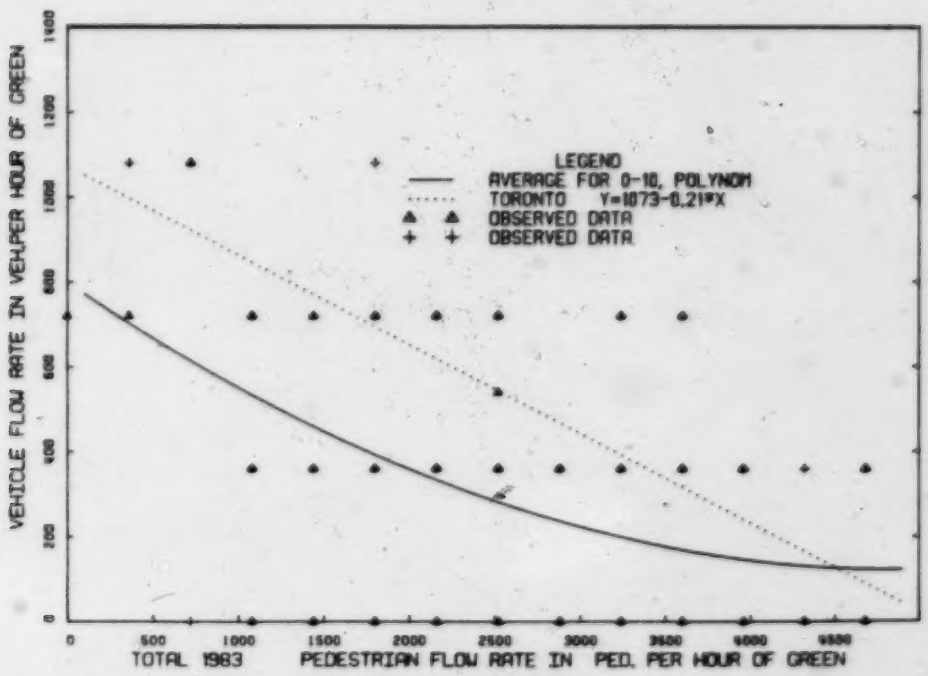


Figure 6.6 Polynomial regression for location #2 (0-10 sec)

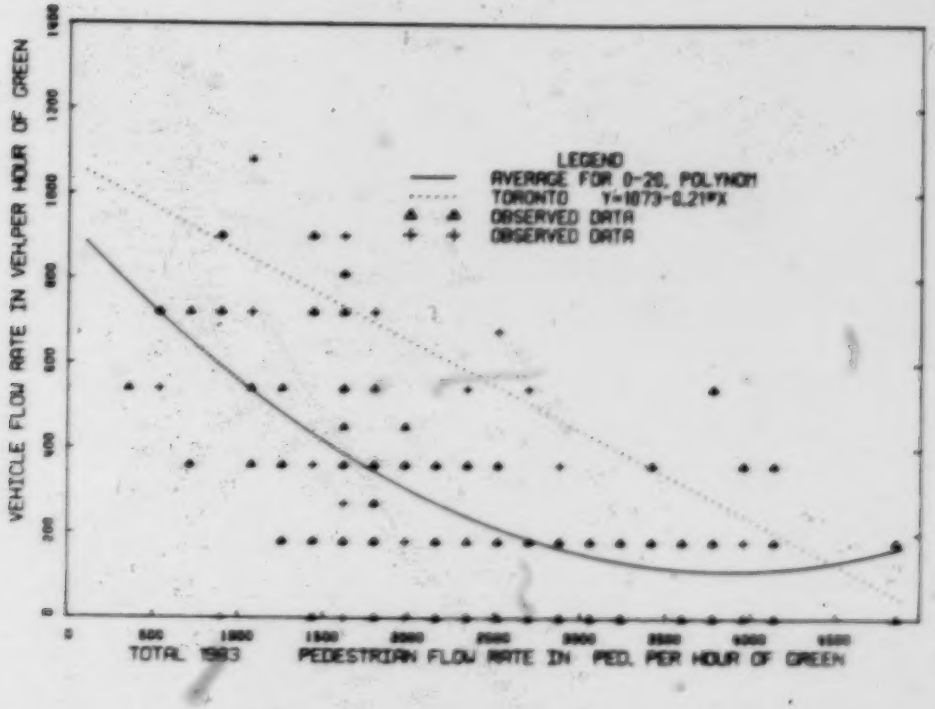

LEGEND
AVERAGE FOR 0-20. POLYNOM
TORONTO Y=1073-0.21*X
OBSERVED DATA
OBSERVED DATA
VEHICLE FLOW RATE IN VEH/PER HOUR OF GREEN
TOTAL 1983 PEDESTRIAN FLOW RATE IN PED. PER HOUR OF GREEN

Figure 6.7 Polynomial regression for location #2 (0-20 sec)

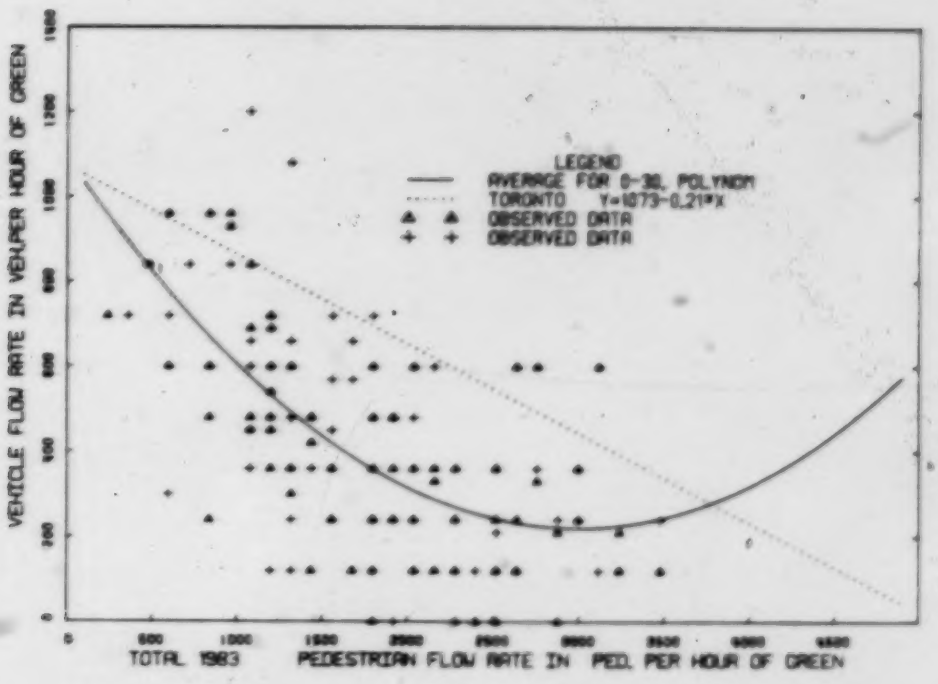

LEGEND
AVERAGE FOR 0-30. POLYNOM
TORONTO Y=1073-0.21*X
OBSERVED DATA
OBSERVED DATA
VEHICLE FLOW RATE IN VEH/PER HOUR OF GREEN
TOTAL 1983 PEDESTRIAN FLOW RATE IN PED. PER HOUR OF GREEN

Figure 6.8 Polynomial regression for location #2 (0-30 sec)

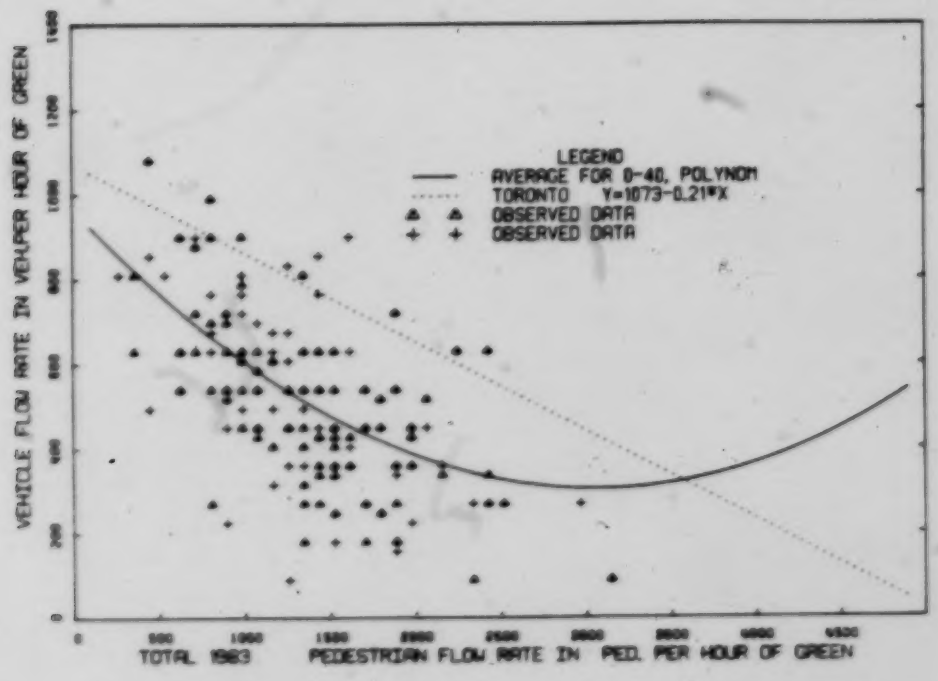

LEGEND
AVERAGE FOR 0-40, POLYNOM
TORONTO Y=1073-0.21*X
OBSERVED DATA
OBSERVED DATA
VEHICLE FLOW RATE IN VEH/PER HOUR OF GREEN
TOTAL 1983 PEDESTRIAN FLOW RATE IN PED. PER HOUR OF GREEN

Figure 6.9 Polynomial regression for location #2 (0-40 sec)

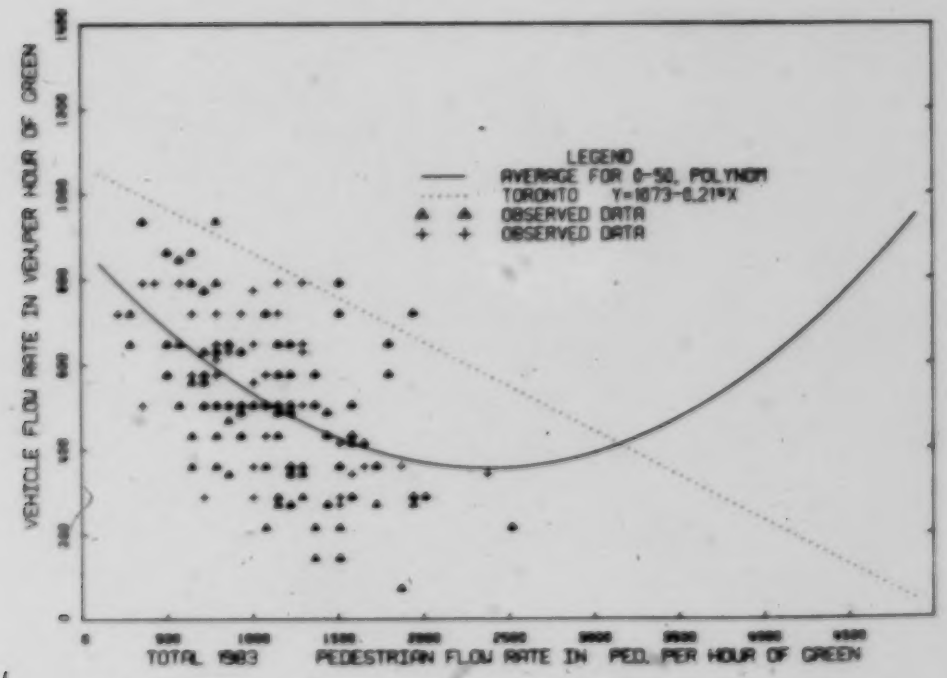

LEGEND
AVERAGE FOR 0-50, POLYNOM
TORONTO Y=1073-0.21*X
OBSERVED DATA
OBSERVED DATA
VEHICLE FLOW RATE IN VEH/PER HOUR OF GREEN
TOTAL 1983 PEDESTRIAN FLOW RATE IN PED. PER HOUR OF GREEN

Figure 6.10 Polynomial regression for location #2 (0-50 sec)

limited data. Evaluation of all data provided also the possibility to confront the results with Yagar's analytical method.

6.5. Summary of the results

The evaluation of location #2, which was selected as the most typical for our case, provided the following results:

6.5.1 Interval 0-10 and 0-20 sec; low pedestrian flow

The conditions with small or moderate pedestrian flow yielded lower vehicle flow rate in 0-10 and 0-20 second slices. The flow should have been higher according to the expectation from the basic concept. This phenomenon was explained to be the result of the influence of the intial stress (see saturation flow concept). The impact of pedestrians creating barrier in the interaction zone was also involved. The bias of different counting methods was negligible in this case.

6.5.2 Interval 0-10 and 0-20 sec; traffic pressure

A different situation occurred when higher pedestrian flow obstructed the vehicle path. Higher values of vehicle flow were observed in 0-10 second intervals and lower in 0-20 and 0-30 second intervals. This effect was caused by the cummulative method of the evaluation. First vehicles sneaked through the initial gap (forced by the traffic

pressure). The following pedestrians from farside stoped the vehicle flow and prevented drivers from penetration. The number of vehicles remained unchanged and the conversion into the flow rate of vehicles/h green caused the drop of flow value. This effect was more pronounced in interval 0-30 seconds.

6.5.3 Interval 0-30 sec

In cases where low pedestrian flow allowed a relatively smooth uninterupted vehicle flow during the initial part of the cycle, the initial stress became eliminated during the duration of the green. Higher vehicle flows were recorded during the 0-30 second slice due to this effect.

6.5.4 Interval 0-40 sec; low pedestrian flows

The time interval 0-40 seconds was influenced by the pedestrian flow, mainly in its total values. It was also found that during lower pedestrian flows this interval had lower vehicle flow than the 0-30 second one. The influence of a bigger gap was partially overhelmed by the higher speed in the case of straight through lanes. The increasing of the speed was impossible for the right turning vehicles as the drivers had to be prepared for a sharp turning manouver. Moreover the anticipation of possible interaction with pedestrians prevented drivers from moving fast.

6.5.5 Interval 0-40 sec; high pedestrian flows

High pedestrian flows decreased the total number of penetrating vehicles tremendously, but the interval of 0-40 sec yielded higher total flows than interval 0-30 sec. So-called "rest" period allowed penetration of vehicles after the pedestrian flow from both sidewalks (P_n and P_f) had cleared the interaction zone. Only the pedestrians (theoretically) violating the rules were present in the interaction zone. Then the penetrating vehicles moved faster and freely.

6.5.6 Interval 0-50 sec

The intervals 0-50 seconds of green were considered to be rather rare. They would increase cycle time and the total delays for vehicles, especially in locations with heavy flows. The green time of 50 seconds was not found in downtown Edmonton. Evaluation of the 0-50 second interval was found unnecessary. The vehicle flows from the 0-40 sec intervals over 50 sec were used in the calculation. The increas in total number of penetrating vehicles was small. The interval 0-50 smoothed and averaged all extremes which occurred in prior intervals.

6.6. Evaluation of the problem

6.6.1 General trend

Individual results from large data sets (from combined files) were obtained for 10 second slices of cycle (0-10, 0-

20, 0-30, 0-40, 0-50). The cumulative basis was used. Regression lines were superimposed (Figure 6.11) and the general trend was found. The results had the natural character and followed the basic saturation flow concept (Ref. 8).

Only small differences were found in the trend as well as in magnitudes between the flows from different time slices. This was the important contribution to the solution of the problem. Particular time slices were used in cases where finding of the basic trend was difficult from the whole sample. The trend pronounced in such a slice was substituted for general vehicle flow characterization.

6.6.2 Polynomial curve

Since the polynomial regressions were easy to use and STPK package available, it was used in the first step of evaluation.

Because of scatter of measured data for very high pedestrain values, regression indicated in many cases an upward trend from about 3000 ped/h green. That would mean that flow of vehicles penetrating pedestrian flow would be increasing as pedestrian flow increase, which is a case that is not logical.

For that reason more logical functions were later investigated and polynomial one was rejected. For this step of evaluation the diverging ends of the regression curves were omitted and illogical trends were excluded.

6.6.3 Pilot project - 1982

The analysis from the year 1982 was presented in Figure 6.12. These results were compared with Figure 6.11 which represents the year 1983. The absolute values from the year 1982 were lower, but the trend and characteristics of the shape were the same. Some differences were found for region 0-2000 ped/h green. This effect was explained to be due to the increased traffic pressure during the University Games. This effect was later observed and determined as a very important factor.

6.6.4 Polynomial expression

The results obtained from the regresion analysis were expressed by a single line and by a final polynomial equation (Y = 916.48 - 0.36854*X + 0.000043115*X**2). Its graphical form is presented in Figure 6.14. Polynomial regression expresed the observed relationship in the middle range (1000-3000 ped/h green) well. The deviation was found to be reasonably small (25-80 pcu/h green).

6.6.5 Length of green time

The different timing conditions evaluated in one location did not change the results significantly. The total green portions of the cycle time 80 and 90 sec were set to 33 and 40 sec, but the difference between green and walk (Δ) was the same in both cases - 15 seconds. The

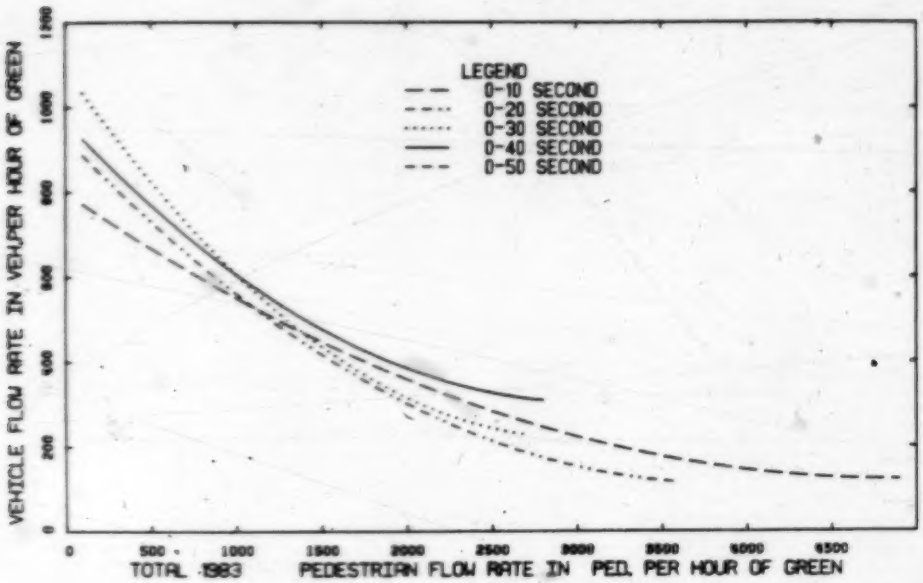


Figure 6.11 Regression lines for individual time slices (1983)

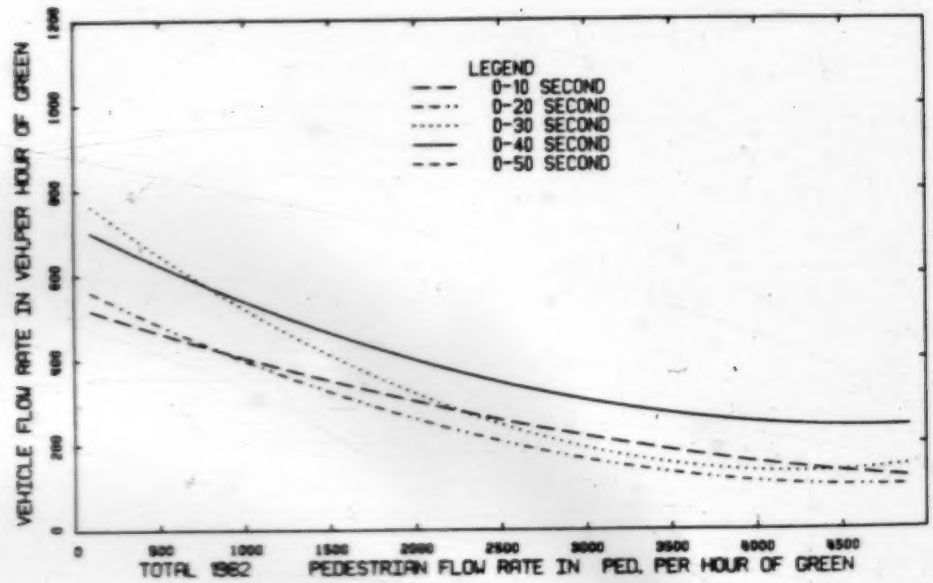


Figure 6.12 Polynomial regression lines representing the pilot project (1981-82)

statement which was derived from this experience confirmed that the slight change in the total green time did not have any big impact on the observed vehicle flow.

6.6.6. Evaluation of derived function

Not many data were available for very low pedestrian flows (range 0-250 ped/h green) in this location. The shape of the polynomial curve was marked by its own character. The range with very high pedestrian flow (over 4000 ped/h green) did not also provide many data and the original shape of polynomial curve was not satisfactory. The additional observations of so-called "high point" and "low point" were necessary. Supporting data were collected and the experience introduced by previous research (Ref. 8) was employed.

6.6.7 Adjustments of polynomial curve

6.6.7.1 High point

The value of vehicle flow rate for pedestrian flow equal to zero was called the "high point" of regression curve. These conditions are rather theoretical for right turning vehicle flows in downtown locations.

The Toronto formula defined the highest point as 1073 pcu/h green. The Edmonton method used the basic saturation flow value. Assuming that correct value of basic saturation flow (as presented in Canadian Capacity Guide - Ref. 8) is 1550 pcu/h green this value describes the highest point of

saturation flow curve. Then for calculation of saturation flow the specific adjustment factors of the location should be applied to the basic value.

This statement was later verified and found to be correct. The difference in the total values between the two methods resulted from the application of other factors such as radius, street width, free vehicle flow etc.

The experience obtained during the observations at a given location indicated that some adjustment factors were applied in limited scale or not at all. Application of the adjustment factors for grade, lane width and in some cases radius were overwhelmed by the strong pedestrian influence. The interference of other factors was then negligible. In the majority of cases the pedestrians forced the driver to come to a complete stop or to slow down the natural free flow.

6.6.7.2 Low point

The value of vehicle flow for conditions with pedestrian flow rate equal to 5000 ped/h green was called the "low point" of the curve. According to the traffic observations it was concluded that for 5000 or more pedestrians/h green the differences of flow were small and yet some flow took place regardless of the pedestrian flow. One or two vehicles passed through the interaction zone during the intergreen period of the cycle. The vehicle flow rate was then 60-240 pcu/h green. The magnitude

depended on the timing conditions (cycle and green time).

In our case the cycle time was 80 and 90 seconds and the green time was 33 and 40 seconds. The average number of penetrating vehicles 1.5 pcu/cycle was used. The low point was established for 5000 ped/h green as 150 pcu/h green. This number was later confirmed to be the average value for all the observed intersections. The vehicle flow had an asymptotically diminishing tendency with the increasing number of pedestrians. The theoretically lowest flow was found to be as low as 60 or 80 pcu/h green.

6.6.8 Additional curve fitting

6.6.8.1 Modification of polynomial curve

A simple parabolic regression curve did not satisfy the above requirements. For this reason an attempt was made to extend the parabolic curve with a straight line running to the lowest point of the Toronto curve (Figure C.24). However this solution did not satisfy the previous requirements on low point value. For this reason the extension with a line having tangential character was attempted (Figure 6.13). This modification did not express the real situation either. The low point for this intersection was observed to be about 100 pcu/h green. The use of the Toronto lowest point was rejected. The regression polynomial curve did not meet the high point requirement. The superposition of the data and regression line in Figure 6.14 shows the real trend. The polynomial

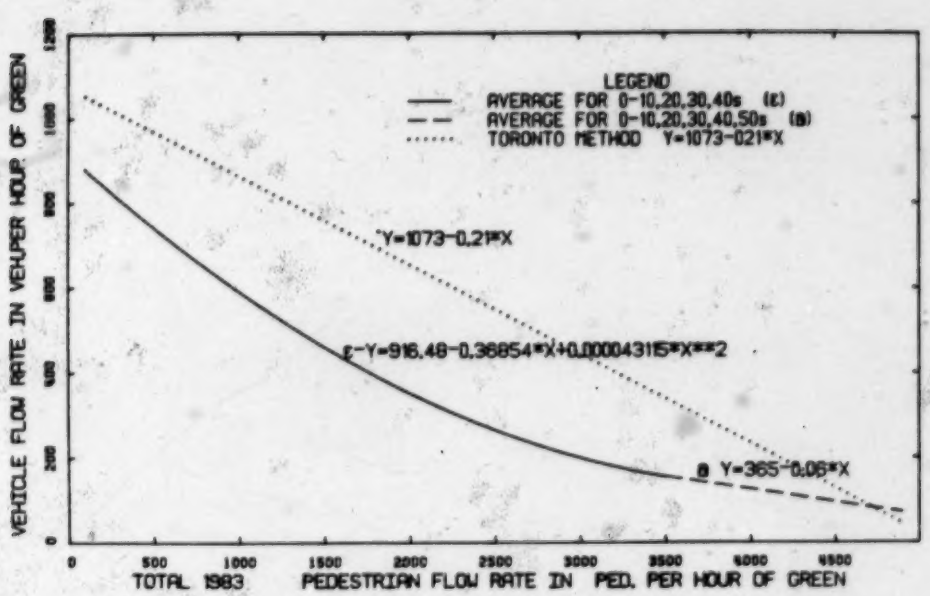


Figure 6.13 Low point adjustment of polynomial curve

slope was not steep enough in the low pedestrian flow range (0-500 ped/h green).

6.6.8.2 Different functions

The application of different types of curves was one possible solution. Another approach was to establish the three regions separately evaluated. The application of the second degree curves better met the thesis requirement for a simple practical solution.

Experiments with exponential and hyperbolical curves are presented in Figure C.15. The combination of exponential and hyperbolical curves brought the good results. However the adjustment factors with flow reducing tendency were hidden in the observed totals. The combination of curves was too complicated. The exponential character was found to be promising for expression of studied relationship. The curve was not complicated and satisfied all regions.

6.6.9 Exponential curve

The simple shifted exponential curve was used to express this step of the evaluation as the most adequate one (Figure 6.15).

The superimposed data in the Figure C.16 substitute the values of the consequently observed cycles. The curve and the formula Y = 100 + 1450/ł**0.001*X expressed the investigated relationship for all the 10 second cumulative

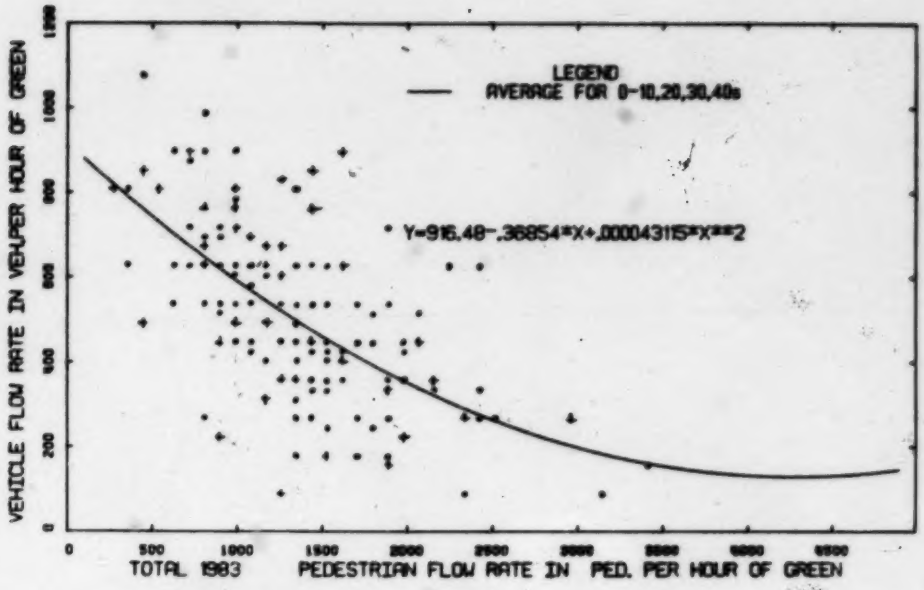

LEGEND
AVERAGE FOR 0-10,20,30,40s
Y=916.48-.36854*X+.00004315*X**2
VEHICLE FLOW RATE IN VEH/PER HOUR OF GREEN
TOTAL 1983
PEDESTRIAN FLOW RATE IN PED. PER HOUR OF GREEN

Figure 6.14 Polynomial expression of the relationship

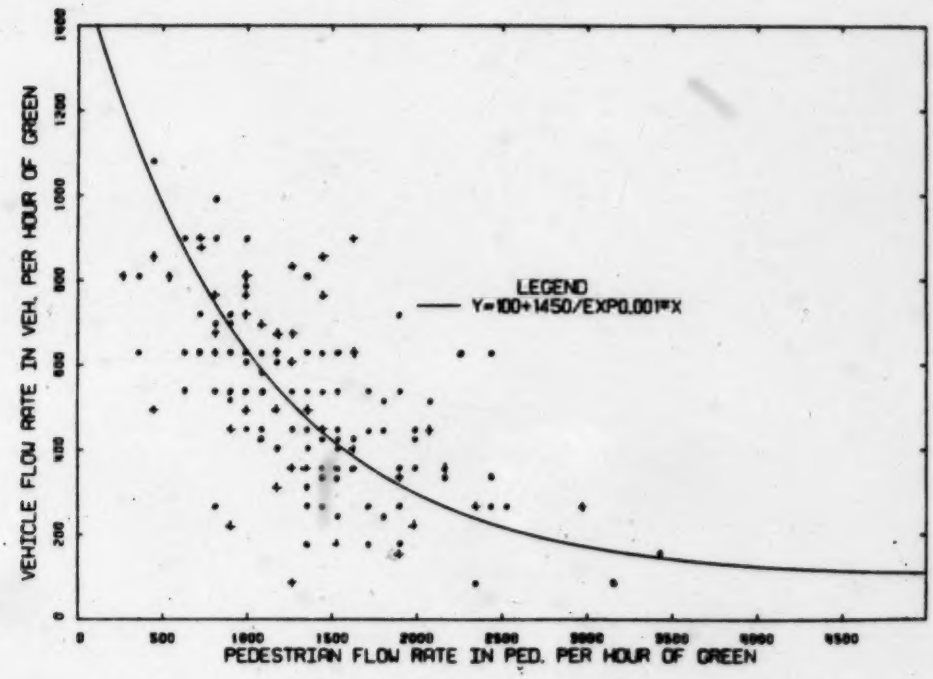


Figure 6.15 Exponential expression of the relationship

time slices.

The need for presentation of the results in The Canadian Capacity Guide suggested the use of the same axes as for the left turn curve. The formula and curve used in the Guide are presented in Figure C.17. The shape similarity with the left turns in the Edmonton conditions was as expected good.

Similarly as for the left turn, the vehicles turned during the intergreen were evaluated separately. The establishing of the curve ends (high and low point) was done with caution, as fluctuation might be expected.

6.7. Verification of the high point

6.7.1 Method

The high point of the curve was determined as a basic saturation flow value for Edmonton. This assumption was derived from the logic behind the saturation flow concept. The next step in dealing with all the locations was possible by using this value as input. The verification of this assumption was necessary.

Observations in the downtown area were concerned with the relationship between the pedestrian and vehicle flows. The presence of pedestrians restricted the observations with regard to the flow of the right turning vehicles during the whole green period. Even one pedestrian interacting during the cycle made the measurement of the saturation flow impossible. The method utilized for this part of the

research provided reasonably confident results.

Following the procedure described in Reference 8, the basic saturation flow was measured and calculated for one intersection out of the downtown area. The location at 97 St. - 122 Ave. yielded one straight through lane without the application of the geometric adjustment factors. The factor for the duration of green was only applied to the traffic conditions.

6.7.2 Location at 97 Street

The measured adjusted saturation flow at 97 street was 1598 pcu/h green. The adjustment factor used for the green interval of 70 seconds was 0.9. The calculated basic saturation flow was found to be 1720 pcu/h green. This flow was slightly higher than the initial flow recommended by the Guide (Ref. 8). The flow of 1650 pcu/h green is generally used for such geometric and traffic conditions - HGS, LA, AM peak (HGS - high geometric standards, LA - low activity). The difference was due to the special conditions of very high flows in the observed location. The maximum or the highest value of the saturation flow was measured in the middle of the cycle. The free flow (rolling start) from this part was calculated as:

$$\text{Free flow} = \frac{\text{number of cars (or pcu)} - 1}{\text{time interval}} * 3600 \qquad (19)$$

The flow measured in this way was 1980 pcu/h green. The

ratio of separated flow over the basic saturation flow was:

$$\frac{1980}{1720} = 1.15 \qquad (20)$$

The procedure of separation or slicing of the interval from the continuous flow affected the volume of the flow. The adjustment factor for the duration of green was not applied to the slices according to the logical consideration.

6.7.3 Location with right turn flow

The same procedure was applied for the downtown location. The average free flow value was determined as 1313 pcu/h green. The "sliced" free flow was measured in right turn lane and radius adjustment factor was applied. The right turning radius was 8.5 m and corresponding factor was 0.72 (Ref. 8). The correct free flow was determined as:

$$\frac{1313}{0.72} = 1824 \text{ pcu/h green} \qquad (21)$$

According to the Guide (Ref. 8) the basic saturation flow recommended for such conditions is used as 1550 pcu/h green. The ratio of the measured over the real flow was then:

$$\frac{1824}{1550} = 1.17 \qquad (22)$$

The application of this indirect method resulted in two

ratios. The ratios did not match perfect, but good enough for the evaluation purpose. The difference between the two ratios was due to the existence of other nonmeasureable factors such as traffic pressure, location specific conditions, etc. Generally the difference was small and the ratios were considered to be the same. This fact confirmed the theory that 1550 pcu/h green was correctly used for the high point of regression curve.

6.7.4 Application of factors

This procedure introduced the necessity for application of the turning adjustment factor for the regions where non or a few pedestrians were involved.

6.8. Conclusion

Development of Edmonton method was carried out at location #2. Measured data at that intersection were successfully approximated with polynomial regression line. However this regression did not describe the searched saturation flow relationship. Further application of exponential curve yielded better results. Differences between these two attempts introduce the involvement of other (than pedestrians) factors. Confirmation of high point of curve logically required to use the radius adjustment factor in case where no pedestrians are present. Other geometric features and traffic conditions were studied and description of result is presented in following chapter.

CHAPTER 7 - EFFECT OF TURNING RADIUS AND TIMING

7.1. Application of the basic curve to other locations

This chapter presents the evaluation of additional factors, especially influence of turning radius and timing.

Further development of Edmonton method was based on data from location #2, which was considered to be typical for downtown area. The other locations were selected with regard to turning radius and timing.

Firstly the data from all the locations were presented in one figure (Figure 7.1). Each point represents 5 subsequently observed cycles. It can be seen that the basic exponential curve is not quite applicable for the best results. The measured data from other locations yield a lower trend. The obvious difference between data and curve indicates the necessity of examining each location closely. The difference in results pointed out the influence of geometric and traffic factors which were different from location #2. The locations were evaluated in accordance with individual factors presented.

7.2. Influencing factors

The factors influencing the vehicle flow were described in Chapter 2 and presented in Table 2-1. The Canadian Capacity Guide (Ref. 8) advises the combination of these factors if they are accumulated. The method of using the factors for right turns however is not determined exactly.

Examination of this case was carried out in a way to eliminate the less important adjustments and to emphasize the important ones. They are:

A. The most important factors influencing right turns are pedestrians, turning radius, the difference between green and walk and length of the crosswalk.

B. Other factors have secondary importance but the selection of locations prevented their application (lane width, total length of green).

C. Factors such as grade, queueing space, bus stops and on-street parking are not involved in this research.

7.3. Influence of turning radius

The obvious factor influencing saturation flow for right turns is the radius. According to the Guide (Ref. 8): "This factor is applied only under severe conditions and affects exclusive right-turn and shared right-and-through lanes. ... Curb radius may be used where applicable; otherwise the inside trailing edge of vehicle should be applied. This factor should be applied with caution if additional adjustments are made for heavy pedestrian volumes." Figure 7.2 (Ref. 8) illustrates the impact of turning radius on saturation flow.

Verification of high point provided in Chapter 6 confirmed the statement from the Guide where no pedestrians were present. Additional observations confirmed other statement, that with increasing pedestrian flows the

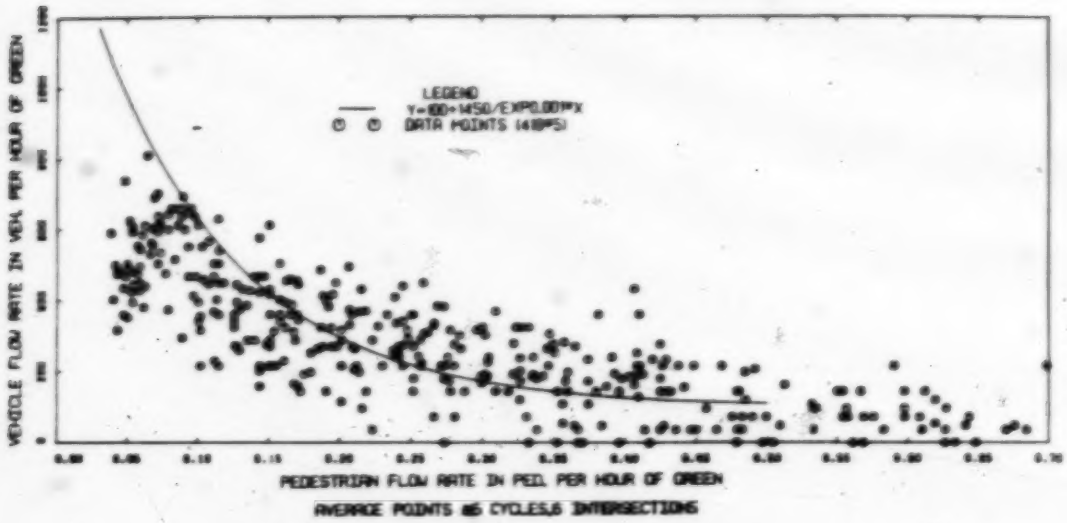


Figure 7.1 Data presentation from 7 Edmonton locations (smoothed at 5 cycles)

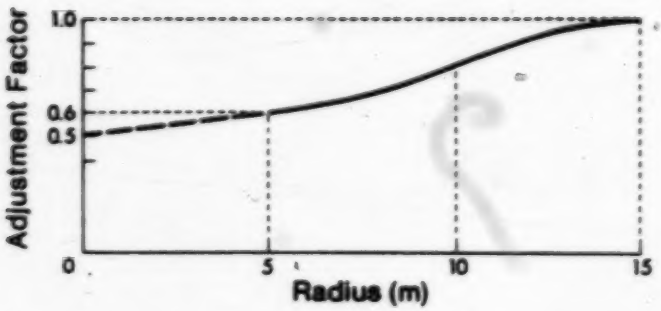


Figure 7.2 Impact of turning radius on saturation flow

influence of radius factor diminishes. In Figure 7.1 can be seen that in region of 2000 ped/h green and more the impact of other factors is negligible (if basic curve is applied). The basic curve represents the measured data for 2000 ped/h green well. With the increasing pedestrian flow the character of curve needs to be raised to express the measured data. It means that the influence of factors lowering the vehicle flow does not apply any longer.

7.4. Analysis of locations #1, #3 to #7

Second degree polynomial regressions were used for each location and each 10 second "slice" to evaluate the trend of relationship between pedestrian and vehicle flows. Regression lines were superimposed for each location to summarize the results (Figures 7.3 to 7.8) and then all plotted together (Figure 7.9). Individual regressions are graphically presented in Appendix C (Figures C.18 to C.47).

The regression equations of the first degree are listed for completeness in Table C-1.

The exponential curve represents the researched relationship well (as described in previous chapter) however the other factors involved (such as radius, timing differences, traffic pressure) required another modification.

The limits or envelopes were established to represent the basic trend (Fig. 7.9) which derived from observed data and describes a direct dependance of vehicle flow on

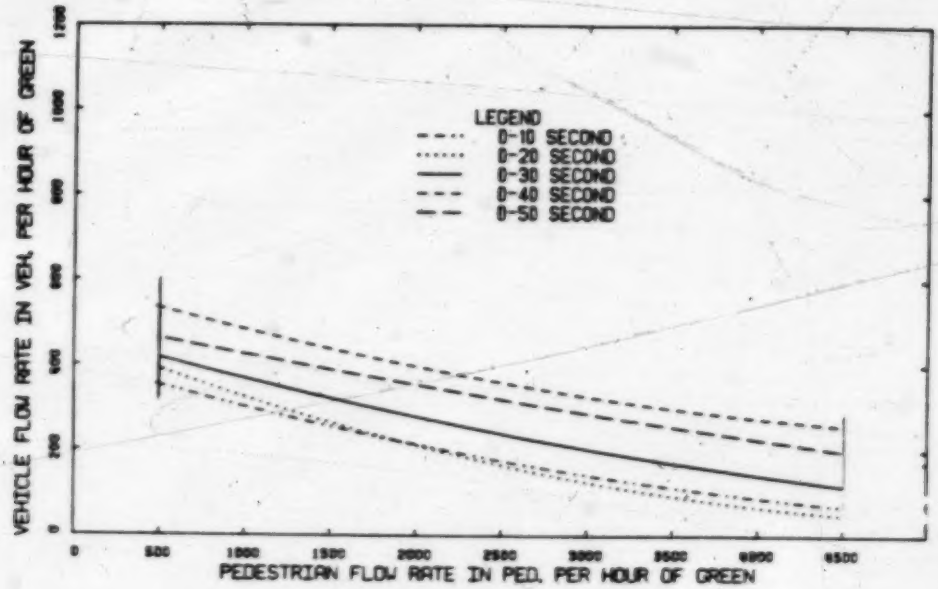


Figure 7.3 Location #1; summary

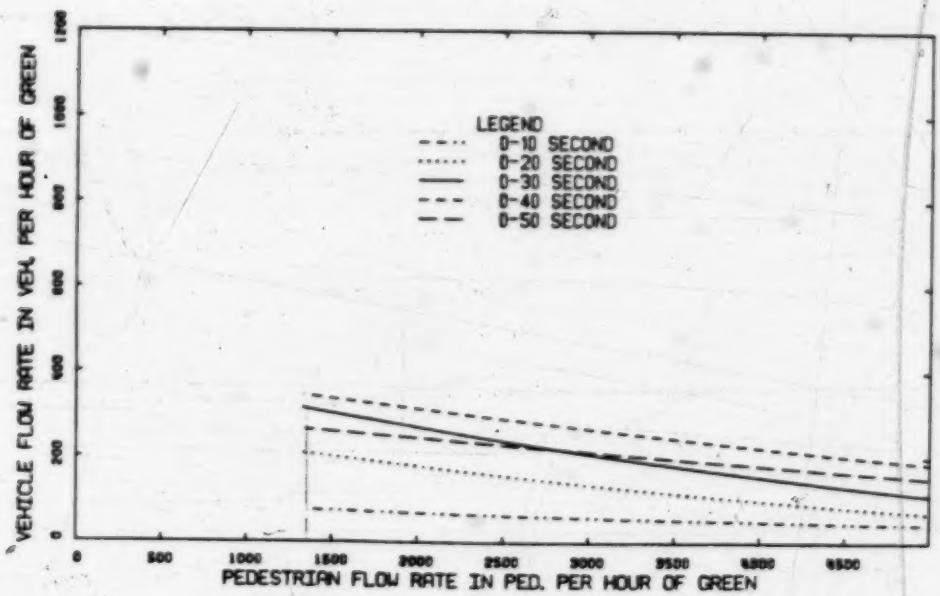


Figure 7.4 Location #3; summary

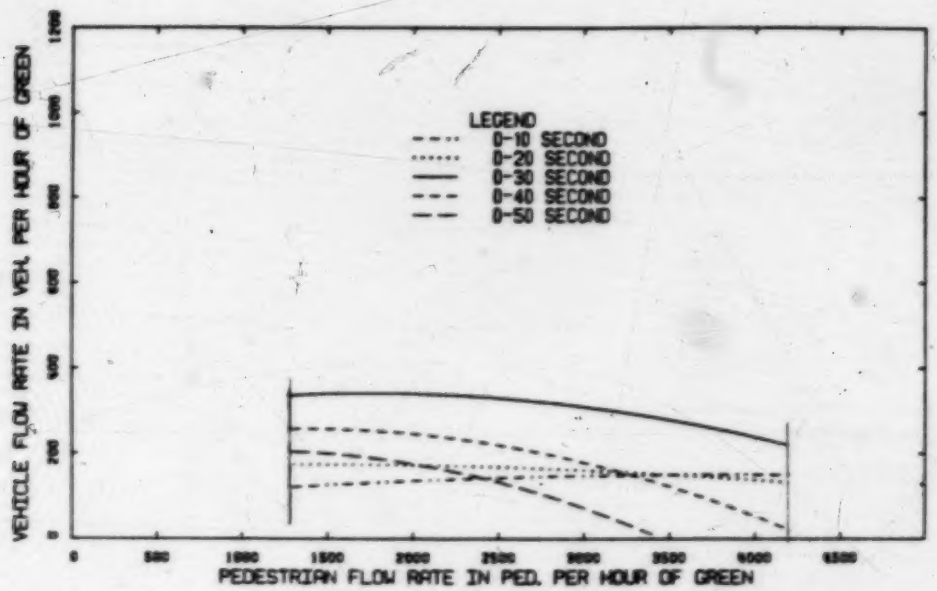


Figure 7.5 Location #4; summary

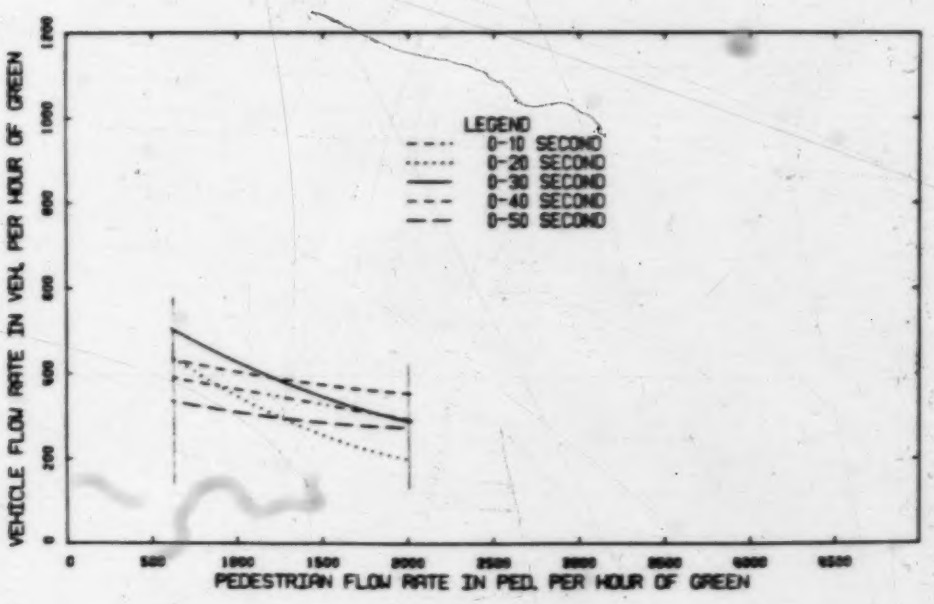


Figure 7.6 Location #5; summary

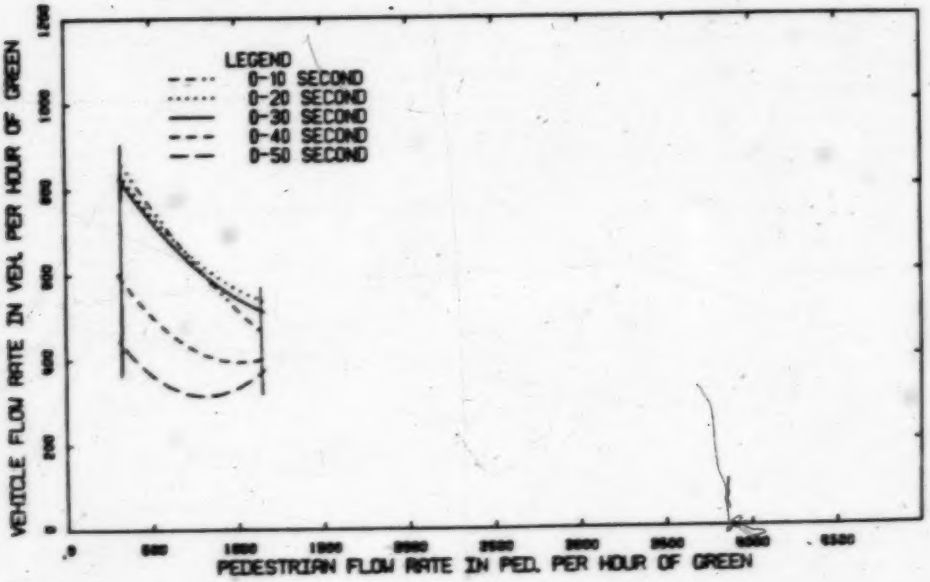


Figure 7.7 Location #6; summary

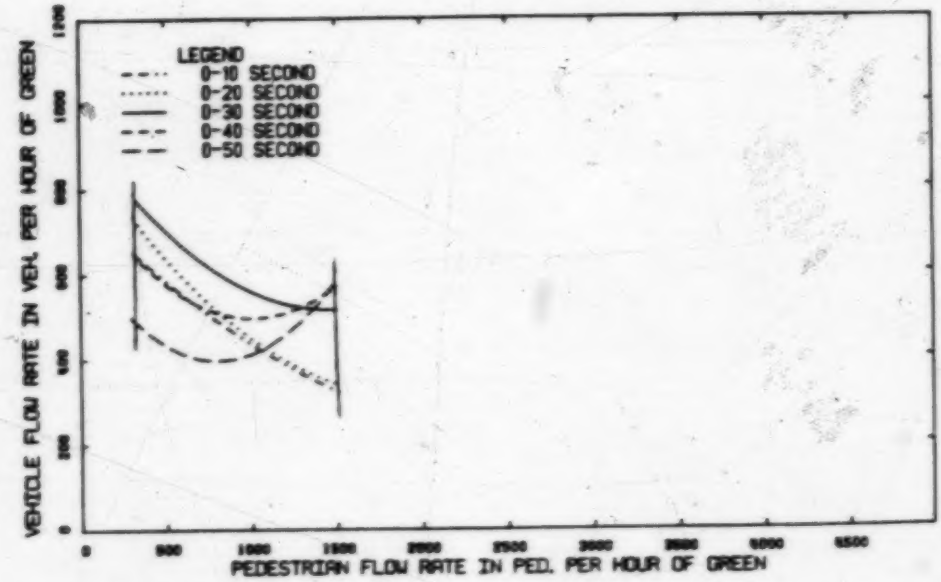


Figure 7.8 Location # 7; summary

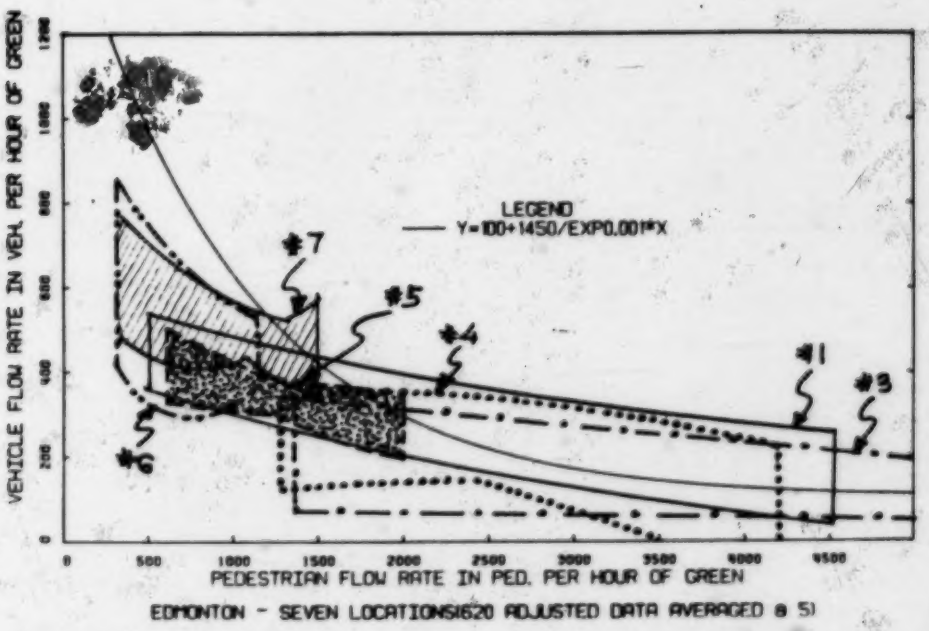


Figure 7.9 Summary of the data regions from all the locations (by using envelopes of regressions)

pedestrians. Differences between envelopes for individual locations pointed to the probability of different factors being involved.

Location #2 is characterized by the basic curve.

Location #1 provides higher values in region 2000 ped/h green and more due to traffic pressure. Lower values in region 500-2000 ped/h green might be due to the sharp radius involved (8.5 m). The region around 2000 ped/h green fits well.

Location #3 provides slightly higher values in region 3000 ped/h green and more due to traffic pressure. Region 1300-3000 ped/h green have lower values because of sharp radius involved (8.5 m).

Location #4 provides almost identical characteristics as location #3. The same radius is involved (8.5 m).

Location #5 has low values in region 500-2000 ped/h green. The total values are slightly higher than in Location #4 for the same radius but it can be considered as a continuation of upper part from location #4.

Location #6 provides higher values than location #5 due to bigger radius (13.0 m).

Location #7 has a radius of 15.5 m and data values are thus higher than for location #6.

According to the result from Figure 7.9, summary of possible impact of factors was generalized:

A. Range 0-2000 ped/h green

Vehicle flow volumes for this range indicate values

lower than for the basic curve. Radius factor and timing factor were involved.

B. Range 2000-5000 ped/h green

The vehicle volumes are determined mainly by the pedestrians. The slight influence of some factors was noticed, especially in the range 2000-3000 ped/h green.

C. Range 5000 and more ped/h green

Vehicle flow fluctuated between 60(or 120) and 180 (or 240) pcu/h green. These variations of flow values were determined by the length of cycle (number of cycles/hour), regardless of pedestrian volumes. The traffic pressure and difference between green and walk interval has also some impact.

This step of evaluation confirmed the statement from the Guide (Ref. 8) that the radius adjustment must be used. Preliminary application of radius factor raises the values in region 0-2000 ped/h green, however in some cases not adequately to meet the general curve. It indicated that other factors were also involved. Possible explanation was found in Yagar's method in which the importance of timing is emphasized. The evaluation of other factors is developed in the next steps.

7.5. Turning radius adjustment factor

The turning radius is one of the factors influencing

strongly the region 0-2000 ped/h green. When no pedestrians interacted with the vehicle flow, the turning maneuver occurred in a natural way, uninterrupted or influenced by the turning radius. Vehicles penetrated freely through the interaction zone with relatively high speed. With increasing pedestrian flow the influence of the radius diminished, being overwhelmed by interrupted flow characteristics (formation of gaps).

The location at 102 Ave.-100 St. S-bound was tested by applying the turning radius factor. For the radius of 8.5 m the adjustment factor of 0.72 (from Figure 7.2) was used in full scale for the case where no pedestrians were present. Zero adjustment factor was applied for the region 2000 ped/h green and more. Linear interpolation of the factor was used between these two extremes. Calculated adjustment factors were applied by multiplication to the values obtained from the basic exponential curve developed in Chapter 6. Calculated theoretical values were compared with measured data (Table 7-1). The results confirmed the correctness of the design procedure.

7.6. Timing

Preliminary application of radius adjustment factor in other locations (#3 to #7) yielded good results and provided the same trend as the basic curve. However some differences still remained and they are presented in Figure 7.10. Individual results of data adjusted by radius factor are

presented in Figures C.48 to C.54. The specific differences between locations are in timing especially in differences between green and walk ($\Delta = G - g$).

Figure 7.10 represents averages of data adjusted by radius factor from locations #3 to #7. Superimposed curve derived from location #2. Differences between green and walk Δ were: 15 sec for location #2; 9 sec for #3 and #4; 11 sec for #5; 8 sec for #6 and 10 sec for #7. The characteristic trend for all locations is present, however the differences in total values remained. Implications of lower values for locations #3 and #4 in region 0-2000 ped/h green is as a result of Δ shorter than for location #2 and #5. Higher totals in region 3000 ped/h green and more are influenced by traffic pressure. Values from location #6 are lower than from #7 and so is the Δ.

It may be concluded that the total flow depends directly on difference Δ.

7.6.1 Length of green

The length of effective green was measured to be about 30 to 40 seconds. According to the Guide (Ref. 8) the adjustment factor for these lengths is 1.0. The results obtained from evaluation of data from location #2 has also shown that the vehicle flow is not very sensitive to the total length of green interval. The difference between green and walk is more important.

PEDESTRIAN VOLUMES	CALCULATED FLOW FROM FIG. 5.8	RADIUS ADJUST. FACTOR	CALCULATED VEHICLE FLOW	REAL OBSERVED VEH. FLOW
0	1550	0.71	1100	1115
500	1000	0.78	780	800
1000	670	0.85	569	580
1500	460	0.925	425	420
2000	300	1.0	350	350

Table 7-1 Example of calculation with radius adjustment factor

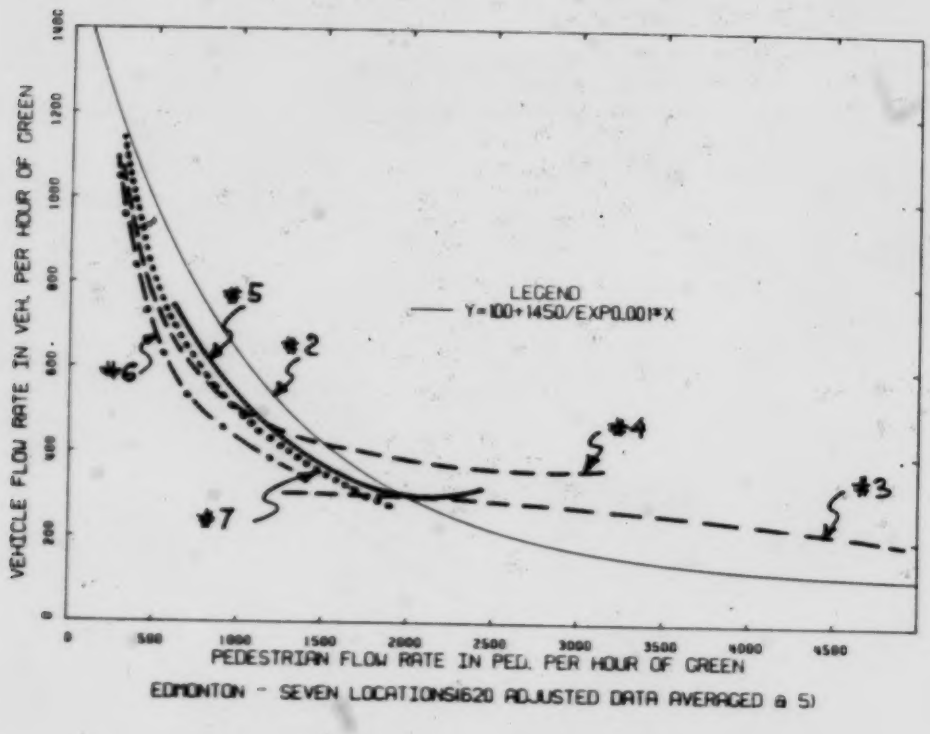


Figure 7.10 Presentation of data adjusted by radius factor (lines representing the averages)

7.6.2 Relationship between green and walk

Comparison between the basic curve and the curve representing the data developed for individual intersections showed that the difference is greater with a shorter difference between green and walk intervals (Δ). Figure 7.11 presents possible function of Δ regulating the free or fully effective right turning flow.

It appears that the duration of the "don't walk" period in excess of clearance period plays as significant a role as that provided by length of crosswalk. The difference between the green interval and the pedestrian walk period (Δ) was found to be 6 to 15 seconds. The "typical" location #2 provided pedestrian clearance period 18 sec which means that Δ equals 15 sec. For this research the difference Δ was used rather than clearance, because the flow of vehicles during and after amber (measured for all the locations to be 3 sec) period is the same regardless of timing changes. A different situation will occur if the green and walk interval are equal, however this case was not included in this research.

The differences between the basic curve and the measured flow were observed for all the locations in accordance with pedestrian flow. Influence of Δ diminishes with increasing pedestrian flow and in region 2000 ped/h green and more is negligible. Table 7-3 presents so-called factor M which was preliminarily estimated as adjustment for

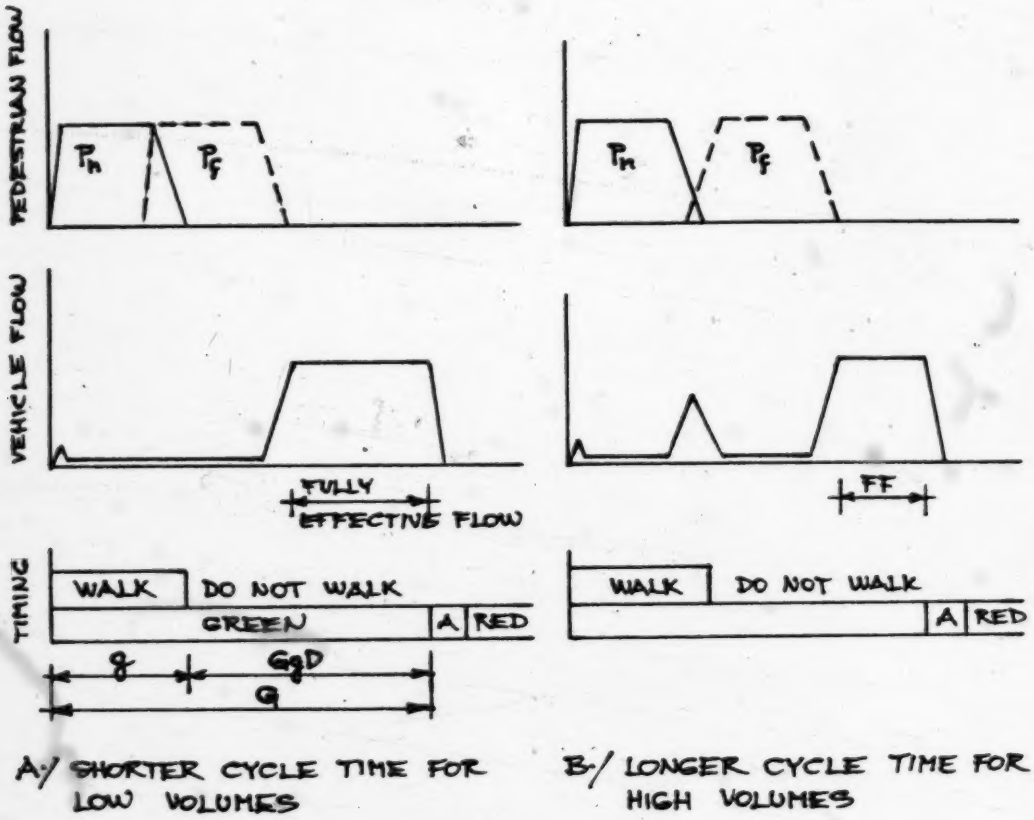


Figure 7.11 Relationship between total green time and free vehicle flow

M FACTOR	PEDESTRIAN FLOW RATE				
	0	500	1000	1500	2000
	100	75	50	10-15	5-15

Table 7-2 Increase factor "M" for each additional second of Δ difference

each 1 second Δ difference. Factor M was substructed from the basic curve values. This rather rough form of adjustment did not express the observed differences exactly and pointed that problem is more complex. Yagar suggested that the factors for timing and length of crosswalk should be combined.

7.6.3 Influence of crosswalk length

All the locations featured a 5 or 6 lane road but with different dimensions, thus the Δ factor cannot be singled out as influence. The impact of the width of the crossed street (length of crosswalk) was difficult to evaluate.

On narrow streets the pedestrians clear the interaction zone faster and thus the time for free vehicle flow is longer. On wide streets the "initial" gap was created and allowed some vehicle flow, but pedestrians did not clear the zone fast and the real free flow was shortened. It was observed that longer "rest" gaps provided higher penetration than the shorter ones combined with the "initial" gap. The worst situation was observed in cases where the street had a medium width. The "rest" time for free flow was shortened with no initial gaps provided.

As previously mentioned the difference between green and walk (Δ) provides the time for pedestrians to clear the crosswalk. The situation at the interaction zone should theoretically follow the example in Figure 7.12.

In that figure the action can be divided into three

time intervals, which are characterized as:

A. No or some vehicle flow (according to the pedestrian flows). - The exponential formula was used for the calculation of vehicle flow.

B. Mixing of pedestrians with vehicles in the interaction zone. - The duration of this period could be expressed by the width of street and the speed of pedestrians as:

$$B = \frac{w}{1.2} \qquad (24)$$

where: B - duration of mixing (sec)

w - width of the crossed street (m)

1.2 - pedestrian speed (m/s)

- the exponential formula was used for calculation of vehicle flow.

C. No pedestrians involved. - Free flow of 1550 pcu/h.

The radius factor should be applied to all three stages. The above description is rather theoretical. The three stages were not observed in real conditions due to several reasons. The most important one was that the length of cycle was never long enough to maintain visibly all three intervals.

In our case the difference Δ was used to express different timing conditions. Observations provided indication that Δ invoked another factor which should characterize the behaviour of pedestrians (responsive

feedback).

Shortening of Δ lowered the right turning vehicle flow for two reasons. Firstly the fact that the mixing period was longer and the total time available for high free flow was shorter. Secondly the psychological pressure was relieved and pedestrian speed decreased due to the longer time available. Consequently pedestrian disobedience increased. Combination of Δ with the total crossed street width was used for better description of the situation. The ratio w/Δ was used as an adjustment factor indication.

It was also observed that with the increasing number of pedestrians the interaction zone was filled more densely. Pedestrians reacted to the situation in the crosswalk and disobeyed the signal display (Fig. 7.13). Density of pedestrian flow depends also on the street width. However for more than 2000 ped/h green the pedestrian flow has a dominating influence and application of timing combined with the above described factors can be omitted.

7.7. Traffic pressure

This adjustment factor was found to be a nonmeasureable item and its influence differed from location to location.

The increasing traffic volumes at the intersection increased the traffic pressure and consequently the number of penetrating vehicles decreased not truly in dependence on the pedestrians. For example the region around 3000 ped/h green yielded higher penetration than expected. It is

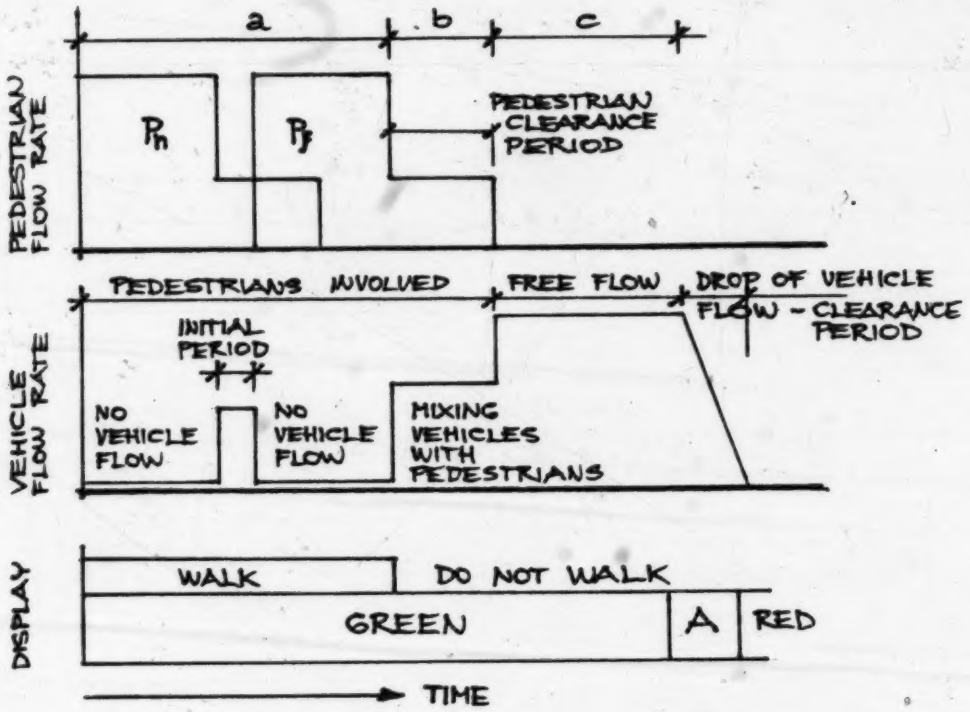


Figure 7.12 Theoretical cases of vehicle flow
a. crosswalk obstructed - no flow
b. interaction with pedestrians - some flow
c. no pedestrians present - free flow

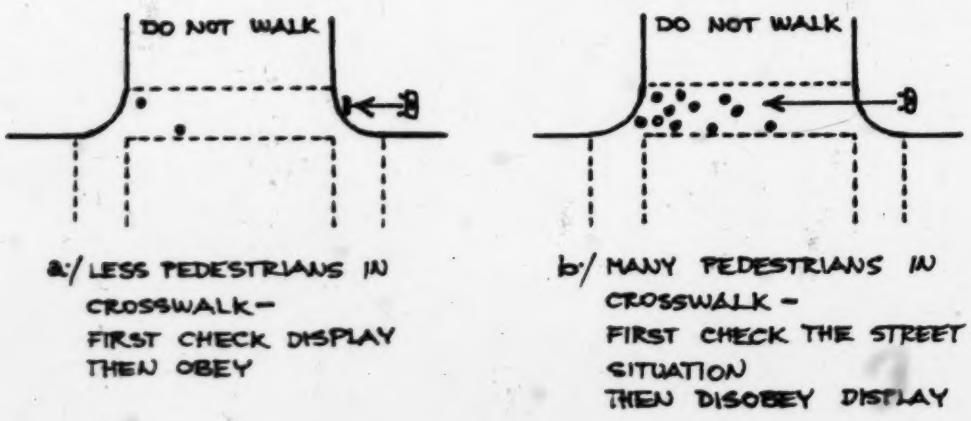

DO NOT WALK
DO NOT WALK
a:/ LESS PEDESTRIANS IN CROSSWALK - FIRST CHECK DISPLAY THEN OBEY
b:/ MANY PEDESTRIANS IN CROSSWALK - FIRST CHECK THE STREET SITUATION THEN DISOBEY DISPLAY

Figure 7.13 Pedestrian discipline at the crosswalk after walk display ended

explained by the fact that in region 0-2000 ped/h green, drivers are waiting for an available gap. With increasing psychological pressure they start to push vehicles into pedestrian flow to force gap creation. With another increase of pedestrian volumes even pushing does not provide a gap and penetration drops. In a few cases the expected trend was not met. Local observation was necessary for establishing of the deviations.

CHAPTER 8 - COMPOSITE EVALUATION

From the previous discussions it is obvious that in the surveyed locations only the turning radius and the difference between green and walk in relationship to width of the crossed street justify the right turning flow which is directed by pedestrians. These characteristics can be summarized in two adjustments called radius adjustment factor (F_R) and flow adjustment factor (S_P).

8.1. Approximations involved

The major goal of the thesis was to develop a practical method. The influence of different factors overlapped and the studied details are included in general form. The practical need of a traffic engineer is not to set the exact number of penetrating cars in each cycle. It would also be impossible to determine the exact value of time biased flow. The interest of the designer or the analyser is to find the expected average volumes which cover all the possible extremes and variations. For this reason the approximation of some factors was made.

Before the development of adjustment factors the approximations involved in the process should be stated.

Program HIS was used to determine the average number of vehicles penetrating the interaction zone in each 10 second "slice". The results are presented in Appendix C (Table C-3) and graphically in Figure 8.1.

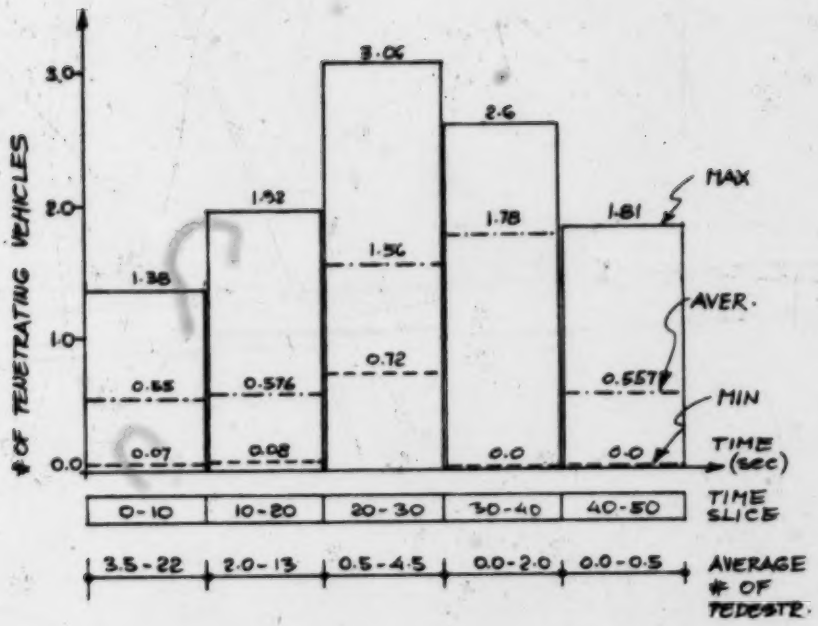


Figure 8.1 Histogram - Number of penetrating vehicles during each 10 sec slice (noncumulative basis)

The right turns depended on many individual factors, as described in the previous paragraphs. The calibration of representative curve dealt with an approximation in the pedestrian volumes. The cumulative method used the highest volumes for expression of saturation flow. Highest volumes were found in 0-30 or 0-40 interval (see Table C-3). The pedestrian volumes were then presented for the same time base. It means that higher pedestrian flows during the walk period were converted into 7 to 15 seconds longer time and used in this form for evaluation. The absolute value used in the calculation was then lower but the error involved is covered by the standards which are used in the evaluation method. The complexity of the problem did not also allow differentiations of each detailed relationship. The overall generalized results covered the possible factors.

8.2 Application of radius adjustment factor (F_R)

Where no other factors are involved, the radius adjustment factor (F_R) should be applied by multiplication for pedestrian range 0-2000 ped/h green directly as presented in the developed nomogram (Figure 8.2).

It was also found that in certain regions (around 3000 ped/h green) the tighter radius increased traffic pressure and penetration of turning vehicles increased. The bigger radius introduced the relaxation of traffic pressure and right turning flow had a tendency to be lower than expected according to the basic curve.

Speculation, that this anomolous behaviour of drivers is due to secondary traffic pressure, exists. However a function for its evaluation was not possible to develop. Presence of this phenomenon can be neglected in evaluation procedure.

8.3 Application of flow adjustment factor (S_F)

For cases where the difference between green and walk (Δ) is 15 sec, no flow adjustment factor is necessary to apply. The influence of uninterrupted vehicle flow is included in the basic exponential formula.

However if the difference Δ is smaller than 15 sec then the combined factor for flow should be applied in the range of 0-2000 ped/h green as presented in the nomogram (Figure 8.3). The value in pcu/h green should be subtracted from the basic exponential curve.

Intersections with Δ bigger than 15 sec were not observed, but in such cases the value of free flow 1550 pcu/h green is recommended to be used beyond that limit.

Multiplicative radius adjustment factor should be used as well if applicable.

8.4 Calibration of basic curve

The data from different locations were all taken and a stated procedure was applied. Computer program FIN was used for computation. A trial and error method was used for fitting of the best descriptive curve. A simple

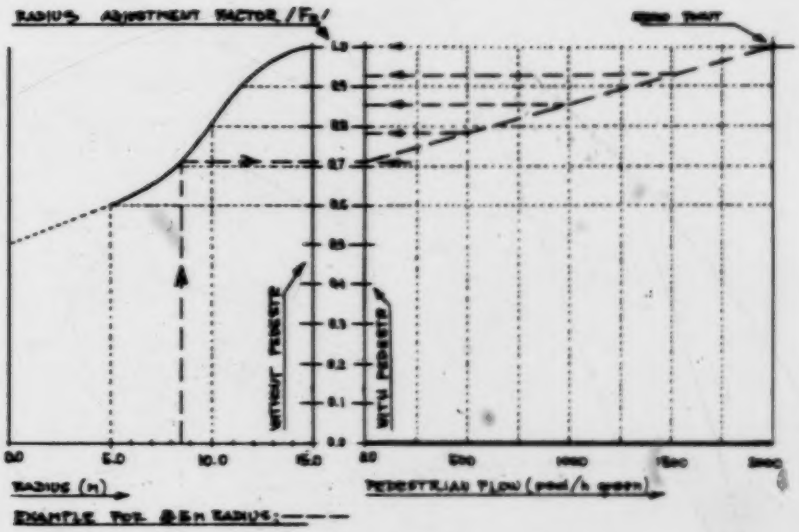


Figure 8.2 Nomogram - radius adjustment factor (F_R)

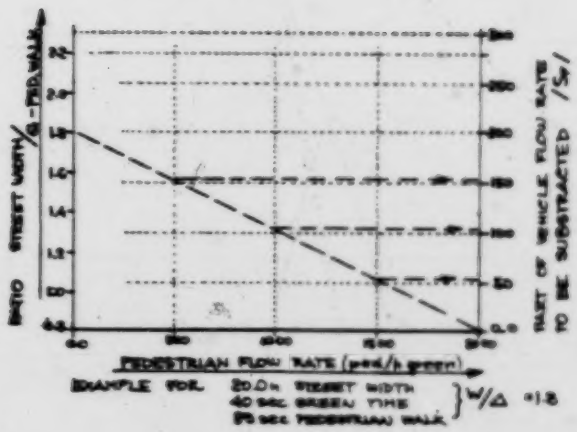

RATIO STREET WIDTH/CROSSED - PED.WALK
PART OF VEHICLE FLOW RATE TO BE SUBSTRACTED /S_F/
PEDESTRIAN FLOW RATE (ped/h green)
EXAMPLE FOR: 20.0 m STREET WIDTH
40 sec. GREEN TIME
W/Δ = 1.8

Figure 8.3 Nomogram - flow adjustment factor (S_F) for differences between walk and green with application of crossed stret width

mathematical expression and the smallest mathematical differences were the objectives. It was found that the basic exponential curve with the following shape was suitable.

$$S_p = 170 + 1380/e^{**0.01*P} \qquad (24)$$

where S_p - saturation flow (pcu/h green)

P - pedestrian flow rate (ped/h green).

Figure 8.4 presents the equation graphically.

Figure 8.5 presents the superimposed curve on measured data. Figure 8.6 presents the superimposed curve on adjusted data.

8.5. Evaluation of confidence limits

Because of difficulties in applying of other methods, the scale x, which describes the pedestrian flow in all figures, was divided into equal slices (@ 500 ped/h green). The data for these individual regions were then statistically evaluated.

It is known that data for such a problem are spread along the regression line in a normal distribution pattern. As said the data from all the locations were sorted into categories according to the pedestrian flow magnitudes. The statistical computer subroutine HIST was used to calculate the mean, variance and the x^2 density function for each category.

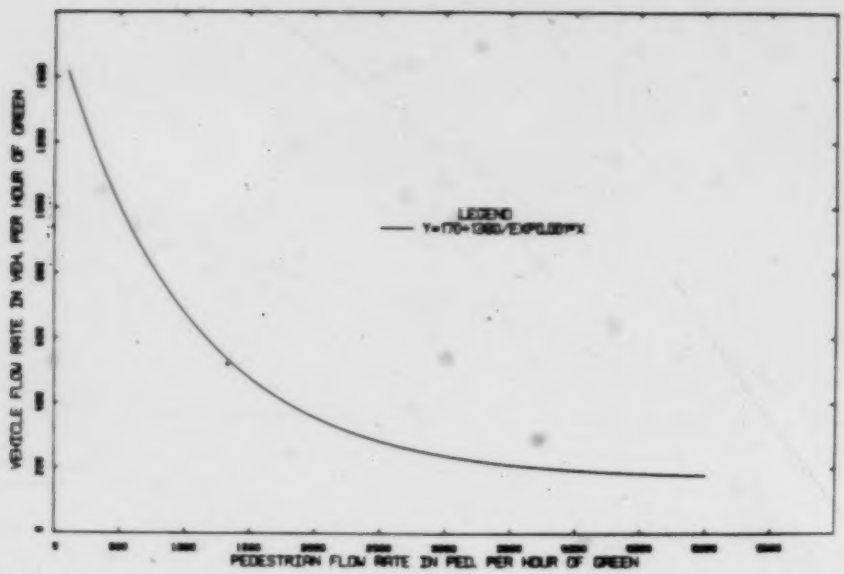


Figure 8.4 Final regression curve

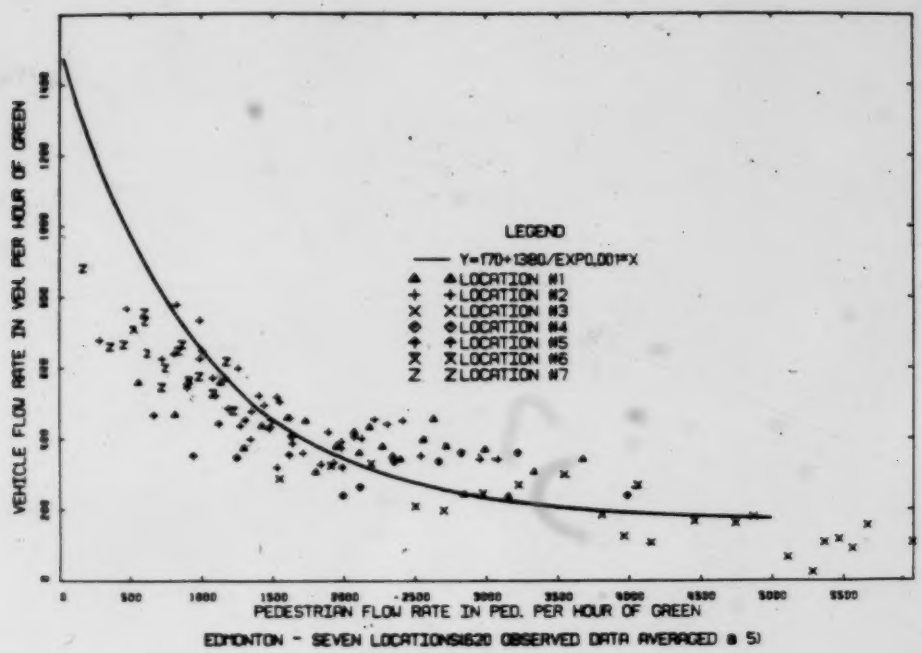


Figure 8.5 Final regression curve and measured data

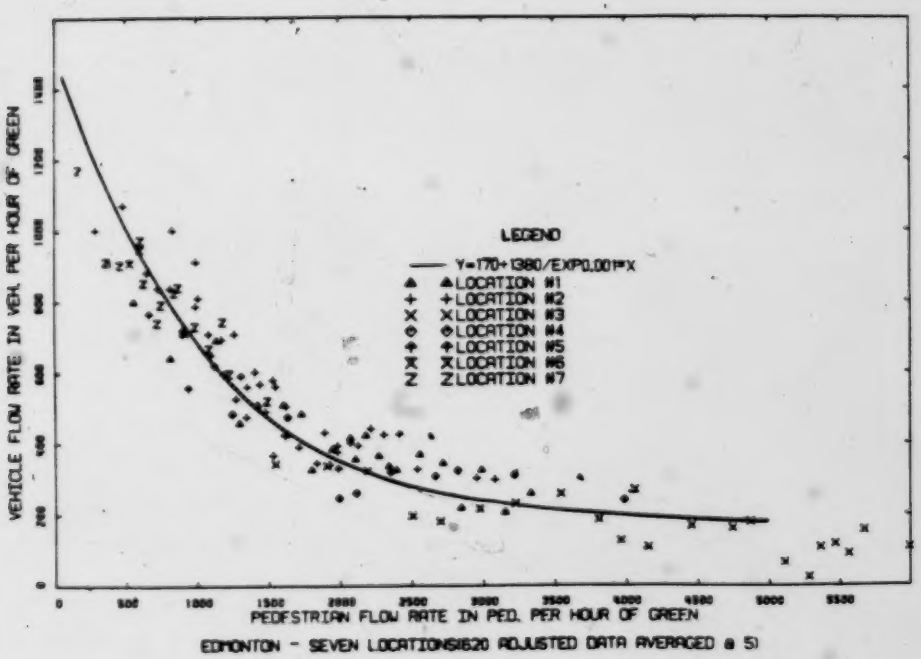


Figure 8.6 Final regression curve and adjusted data

An example of the computer output is presented in Figure C.59. Measured and adjusted data were evaluated separately. The results are presented in Table C-5. Figure 8.7 presents the final regression line superimposed on a bar chart. The calculated means for the observed data are presented by circles. The means for the adjusted data are presented by dots. Upper and lower limits of the mean are presented for each category by dashed lines. These limits represent a 95% significance level. Table C-6 contains their numerical values. The values were calculated by using the following procedure:

A. given mean $\bar{x}$ and standard deviation σ of the sample

B. chosen 95% confidence level; $C_{95} = 1.96$

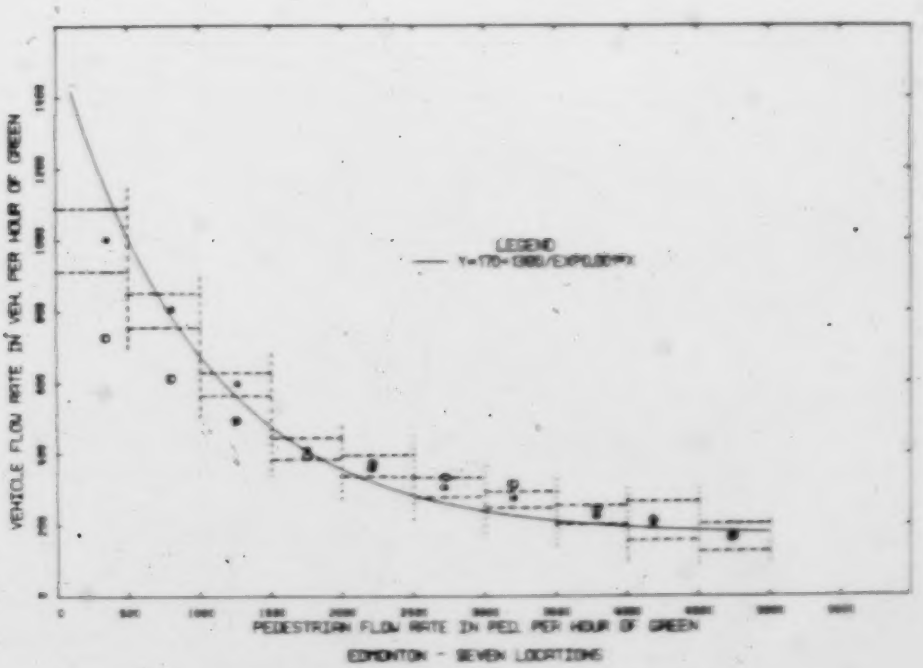
C. computed $k = c\sigma/\sqrt{\mu}$ ' μ - number of data in sample

D. limits are $\bar{x} \pm k$

These limits for adjusted data can be approximated by exponential lines as presented in Figure 8.8. The final exponential curve is superimposed for better orientation.

8.6 Recommended procedure for evaluation of right turns

Based on my experience during this research the following technique of adjustments for right turning vehicle flows as interfered by pedestrians is recommended if combination of influencing factor is encountered. This procedure might eventually complete the appropriate section in Canadian Capacity Guide for Signalized Intersections (Ref. 8).



o means of measured data
• means of adjusted data
--- 95% confidence limits for adjusted mean

Figure 8.7 Distribution of adjusted data

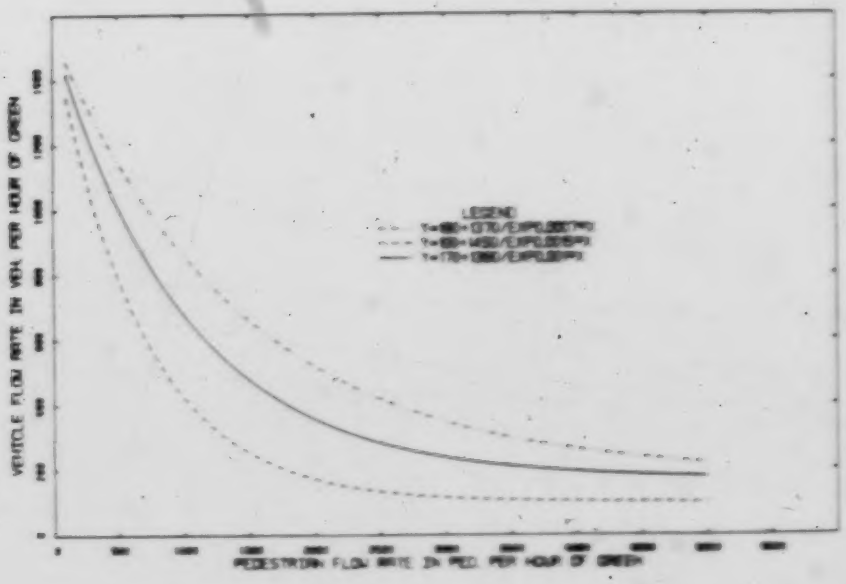


Figure 8.8 Limits for 95% confidence

Saturation flow should be calculated by using formula

$$S = (S_p - S_P)*F_R \qquad (25)$$

where: S - adjusted saturation flow (pcu/h green)

S_p - saturation flow as influenced by pedestrians (pcu/h green)

S_P - saturation flow difference as influenced by Δ factor and width of the street (pcu/h green) - if applicable

F_R - radius adjustment factor (nondimensional) - if applicable

8.6.1 Determination of right turning saturation flow

The following is applicable for region 0-5000 ped/h green. Beyond this region slightly lower values will be applied.

$$S_p = 170 + 1380/e^{**}0.001*P \qquad (24)$$

where: S_p - basic right turning saturation flow (pcu/h green)

P - pedestrian flow (ped/h green)

8.6.2 Determination of radius adjustment factor

This factor is applicable in region of 0-2000 ped/h green. The nomogram (Figure 8.2) can be used according to

the radius and pedestrian flow.

8.6.3 Determination of flow adjustment

This factor is applicable in region of 0-2000 ped/h green. Nomogram (Figure 8.3) can be used according to the difference between green and walk interval (Δ) together with width of crossed street (length of crosswalk) as well as pedestrian flow.

8.6.4 Determination of other factors

In special cases, where for example extreme traffic pressure is observed or where high platooning takes place, the final computed values can be slightly raised or lowered. This factor is applicable in all regions with caution.

8.6.5 Discussion of results

Program FIN was used for calculation of adjusted data by application of designed procedure. Each measured data was then in pair with the theoretical one. Both types of data were presented graphically, measured in Figure 8.9 theoretical in Figure 8.10, and approximated 95% confidence limits were superimposed.

As can be seen the designed procedure yielded the results fair for practical use. The procedure is presented in more detail in Appendix D.

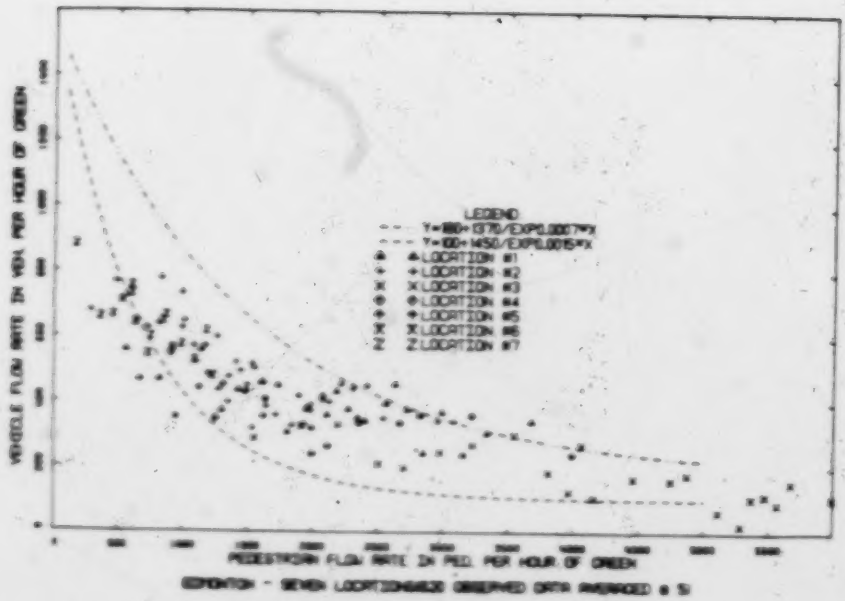


Figure 8.9 Measured data from all the locations averaged in regions (before adjustment)

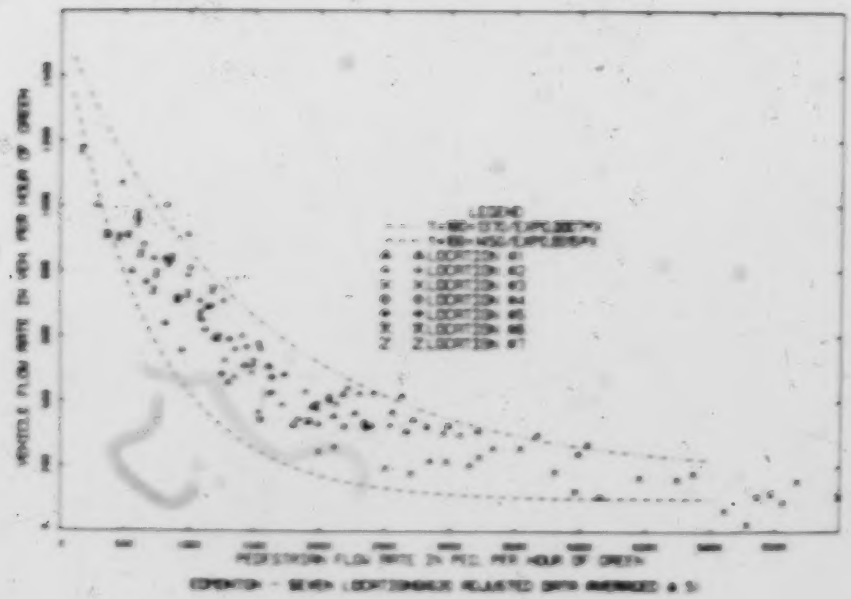


Figure 8.10 Data from all the locations (after adjustment)

CHAPTER 9 - CONCLUSIONS

Several conclusions can be drawn from the research experience.

9.1 Limitations involved during the research

A. All data were collected from Edmonton locations.

B. No location featured gradient.

C. No location needed application of a lane width adjustment factor.

D. Parking interference and obstructions of approach and discharge lanes were not included in research.

E. All unusual conditions such as congestion or straight-through traffic were excluded.

Findings can be listed as follows:

A. Interaction zone was longer than that proposed by Yagar generally as a width of two lanes. Drivers preferred utilization of second discharge lane.

B. Obstructions (bus stops, parking) of approach and first discharge lanes did not influence the vehicle flows if they were far enough from the street corner.

C. Pedestrians and drivers behaviour was somewhat influenced by a combination of pedestrian and vehicular volumes. The factor called traffic pressure could not be quantified but it appears that it should be considered.

D. Modified surveying method used counting point at one third of interaction zone instead of stop line (as proposed by Guide - Ref. 8). The results were satisfactory. Division of timing cycle into 10 second "slices" was proper and accurate for the purpose of this research.

9.2. Comparison of the three available methods

A. Toronto method provided results higher than field data observed in Edmonton where pedestrian flows of 1500-3500 ped/h green were involved. Due to a linear characteristic of Toronto regression line, the results were low if pedestrian flows of 0-1000 ped/h green and 4000 ped/h green or more were involved.

B. Yagar's method provided good results in case of pedestrian flow higher than 2000 ped/h green. For the flow of 0-1000 ped/h green the trend illogically indicated increase of saturation flow with increasing pedestrian flow.

C. Edmonton exponential function yielded promising results satisfying the logical trend of right turning saturation flow for all values of pedestrian flow involved.

9.3. Data evaluation and further development

A. Evaluation of data needed computer processing and

graphical presentation.

B. Polynomial regression analyses were useful for trend evaluation.

C. Analysing of small data sets was sometimes misleading, especially for extrapolation beyond the data sample.

D. Analyzing of averaged data was useful if sufficient amount of them was available.

E. The shifted exponential curve best expressed the researched problem. It was simple and described right turning saturation flow as affected by pedestrians (basic conditions) without additional influencing factors.

9.4. Adjustments for radius and signal timing

A. The statement from the Guide (Ref. 8) can be justified. Radius adjustment factor should be applied in full scale for zero pedestrians involved. Its influence has diminishing tendency with increase of pedestrian flow. Beyond the volume of 2000 ped/h green the impact of radius was negligible. Linear interpolation can be used.

B. For typical 4-6 lines conditions it was found that the difference between green and walk intervals (Δ) can be used as a substitute for extended pedestrian clearance period as an expression of complex relationship among the width of the crossed street,

pedestrian clearance period and intergreen. This factor is used in proposed procedure for additional right turning saturation flow adjustment factor.

C. All not quantifiable factors of local and regional conditions should be also considered (i.e. traffic pressure, type of community, location of the intersection in downtown or outside, type of peaks etc.).

9.5. Proposed method

New modified technique or procedure for evaluation of right turn saturation flow with pedestrian interference is proposed as follows:

A. New formula suggested for saturation flow influenced by pedestrians is:

$$S_P = 170 + 1380/EXP\ 0.001*P \qquad (24)$$

where: S_P - basic right turning saturation flow (pcu/h green)

P - pedestrian flow (ped/h green)

This formula is graphically expressed in nomogram (Figure 8.9).

B. Adjustment factor (F_R) for radius is proposed in nomogram (Fig. 8.2).

C. Adjustment factor for flow difference Δ (S_P) is suggested in nomogram (Fig. 8.3).

D. Calculation of right turning saturation flow (S) with combination of all factors is suggested by using formula:

$$S = (S_P - S_F) * F_R \qquad (25)$$

E. In cases where other factors have severe influence, they may be considered.

F. Proposed procedure was successfully tested by using data from seven locations.

9.6 Further research needs

The proposed method was verified in Edmonton conditions with good results. Due to different environmental conditions in other cities I would recommend further verification and calibration of the method at different locations and cities.

The new method includes the turns during intergreen. It appears that this interval warrants more detailed investigations. Right turn-on-red has not been included in this project. Since it may constitute an important portion of intersection capacity, it should also be researched.

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APPENDIX A - FIELD OBSERVATIONS

This appendix presents the saturation flow surveys and conditions suggested not to be typical or standard for the topic of the thesis. Some of them were hard to evaluate (i.e. traffic pressure) but their importance required that they be studied.

A.1. Basic saturation flow

Previous research in Edmonton has proved that the saturation flow in certain locations remain unchanged, unless the conditions are changed. The stability of saturation flow is presented in Figure A.1.

A.2. Traffic characteristics - traffic pressure

This phenomenon was defined in Chapter 4 (Paragraph 4.4.4). The factor of traffic pressure influenced the right turning vehicle flow significantly.

The utilization of the interaction zone depended not only on geometrical factors but also on the character of traffic conditions. Four examples of turning movement were generally found.

A. A usual example was found when traffic was rather low or medium and volumes of pedestrians were also medium or lower. No "traffic pressure" was present, the driver waited for a longer gap and generally preferred a bigger radius by using the second lane (Fig. A.2).

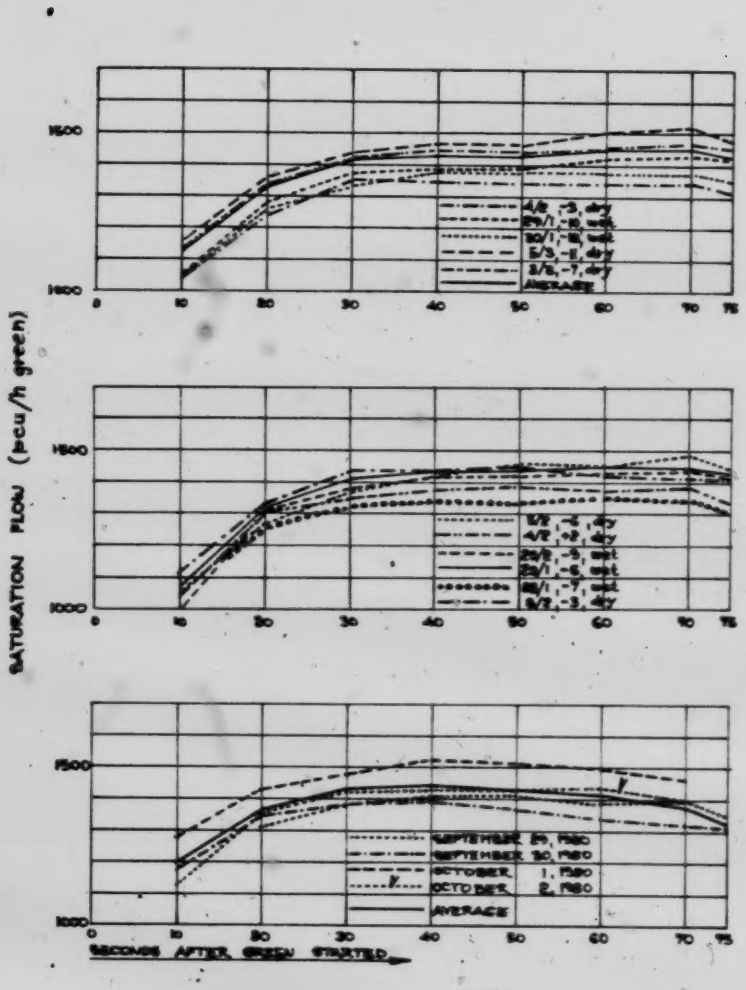


Figure A.1 Saturation flow surveys

B. A different situation occurred when traffic was still medium, but pedestrian flows were high, especially in the case of high farside flow. Psychological pressure of possible obstruction in the form of pedestrian flow forced the more aggressive driver to use a smaller radius and penetrate into the first lane through pedestrians randomly distributed in the interaction zone (Fig. A.3).

C. When the traffic conditions were severe the second lane became, in the majority of cases, obstructed by queueing vehicles. In this situation the traffic pressure mobilized the driver to solve the situation by using the suitable space in the first, or more often, in the third lane (Fig. A.4).

D. Moderate pedestrian flows and very high vehicle flows in both directions created the case where the driver is psychologically forced to violate traffic rules and utilize the double space (Fig. A.5). This situation was found to be similar to the double left turns except that its occurrence is not desirable. From a safety point of view such conditions required special consideration. For example, an advanced phase for right turning vehicles, or different loading of the network should be used.

E. A combination of these previous situations was also detected. The most critical situation occurred in the case of high vehicle and high pedestrian flows. The traffic psychological pressure increased aggresivity of drivers. The first vehicle moved into the first half of the

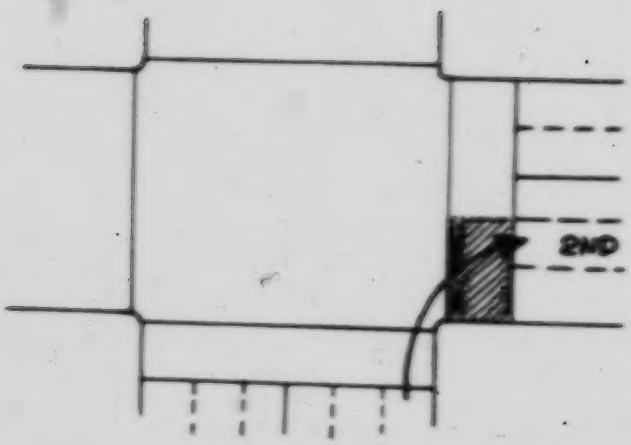


Figure A.2 Typical situation in the interaction zone

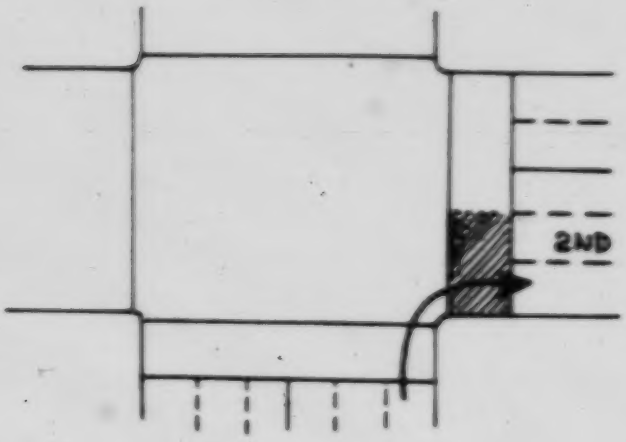


Figure A.3 The case of small traffic pressure (in recorded lane)

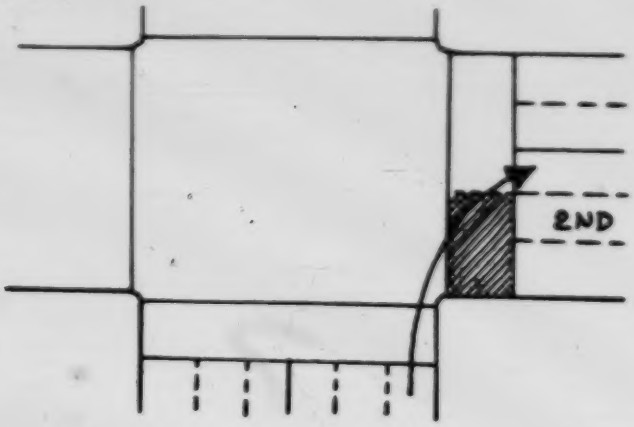


Figure A.4 The case of obstructed second lane (by vehicle queue)

interaction zone "pushing" the pedestrians randomly leaving the nearside curb. They were forced to utilize the second half of the crosswalk width only. This maneuver allowed the opportunity for the second vehicle to pass and wait for a gap in front of the third lane. Dispersed pedestrians from farside formed a new dense platoon in front of the first vehicle and found their way to cross the street. Randomly arrived pedestrians increased their speed in an effort to reach the first platoon. The described action created a situation beyond the intersection's capacity and can be described as a yield point similar to that in Hooke law (Fig. A.6).

F. The right turning vehicle flow generally decreased with increasing pedestrian flow. However in some cases high pedestrian volumes increased traffic pressure. Then, as a result of pressure the declining tendency of right turning vehicle flow changed into an unpredictable phenomenon. The vehicle volumes fluctuated and were high enough to be in contradiction with the predicted magnitude.

G. A turning radius factor is generally applied to turning vehicle flow as an adjustment factor. The application of this factor was correct in cases where no pedestrians were present. Nevertheless its influence diminished with the increasing pedestrian volume which forced vehicles to stop or slowly penetrate the interaction zone regardless of the radius. Traffic pressure created situations where, for the same pedestrian flow, more

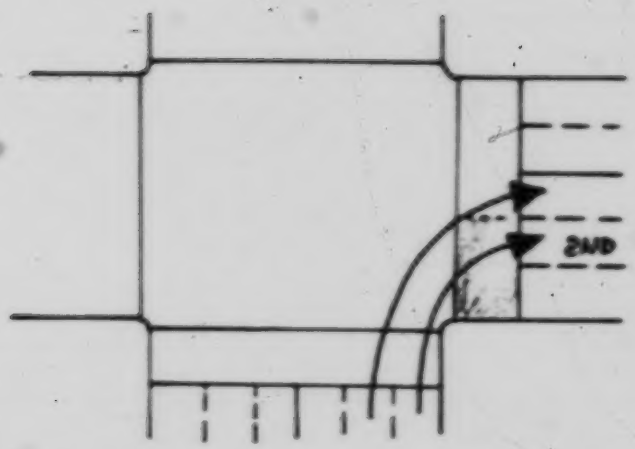


Figure A.5 The case of double right turns

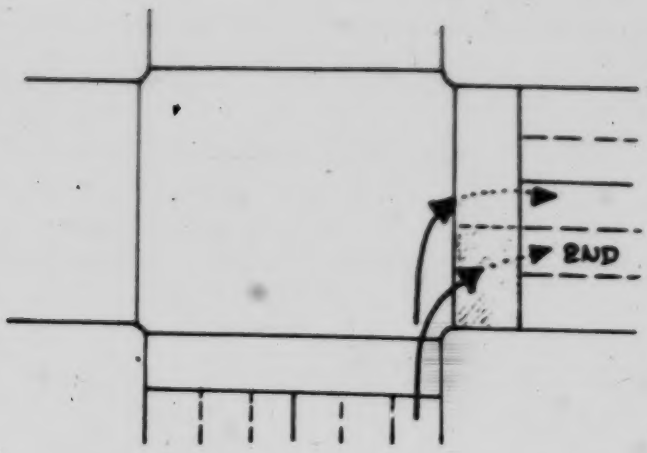


Figure A.6 The case of violation due to high traffic pressure

vehicles penetrated by using a tighter radius. This phenomenon, apparently illogical, is then fully explanable. Unfortunately the traffic pressure is a mathematically indescriptable factor.

A.3. Special conditions

A.3.1 Congested situation

The congested situation did not allow the use of any model. The drivers were willing to violate the rules as a result of extreme traffic pressure. They used lanes without order, blocked crosswalks, blocked the vehicle flows from other directions, etc.

A.3.2 Under-saturated conditions

When the pedestrian volumes were too small the initial platoon of limited number of people crossed the interaction zone. After that free flow of vehicles occurred. The longer the green time lasted the more pronounced free flow occurred. The measured volumes were characterized as a basic saturation flow adjusted by geometrical and timing factors. When the vehicle flow was low, the interaction with pedestrians did not occur, regardless of pedestrian volumes. These cycles were excluded as they did not govern the research problem.

A.3.3 Obstructions

A.3.3.1 Obstruction of discharge lanes

A. Vehicles, after passing the interaction zone, normally utilized the second lane. In cases where the second lane was obstructed, drivers used the third one for discharge. They prefered this solution to the utilizing of the first lane (Fig. A.4).

B. The situation was handled differently where the street had only two lanes for discharge. As before, the second lane was drivers' first choice. If this lane was obstructed, the vehicles were forced to use the first lane, even when space was limited. Although this situation changed the geometrical conditions, the space left behind parked buses was large enough to accumulate vehicles from one cycle. In this case the flow of right turning vehicles was not significantly influenced by obstruction (parked buses).

C. Obstruction of the first discharge lane is very common for downtown areas. Frequent stops of service vehicles or buses were observed. For most cases, the right turn was not influenced at all. Where obstructing vehicles were large enough to decrease drivers' range of vision by parking close to the crosswalk, the first turning vehicle experienced a negligible delay from a shock-like behaviour of the driver (Fig. A.7).

A.3.3.2 Obstruction of approach lanes

A. Other cases of obstruction were found in the first approach lane. As before, parked service vehicles or buses were involved. The waiting point for right turning vehicles was generally the stop line. Where the obstruction was far enough from this point, the vehicles formed a queue of 4-5 units long during the waiting period. Later (green interval), during the discharge other vehicles formed the second lane. After passing the obstruction they were added to the queue from the second lane (Fig. A.8).

B. Mixed traffic in the second lane (straight through and right turning vehicles) generally caused the formation of full queue in that lane and made filling of storage more difficult. However, right turning flow was delayed by interacting with pedestrians and storage was almost never emptied (Fig. A.9). In such a situation no additional (parking interference) factor was applied.

C. Obstructions at waiting points completely changed the traffic conditions. The right turning vehicle flow in the first lane came to a complete stop until the obstruction was removed. In the case of permanent or long lasting obstruction, the right turning vehicles tried to overcome the problem by mixing with the flow in the second lane. Due to the difficulties of formulating such maneuvers and complexity of factors involved, no rules were formed. Such observed cycles were excluded from this research (Fig. A.10).

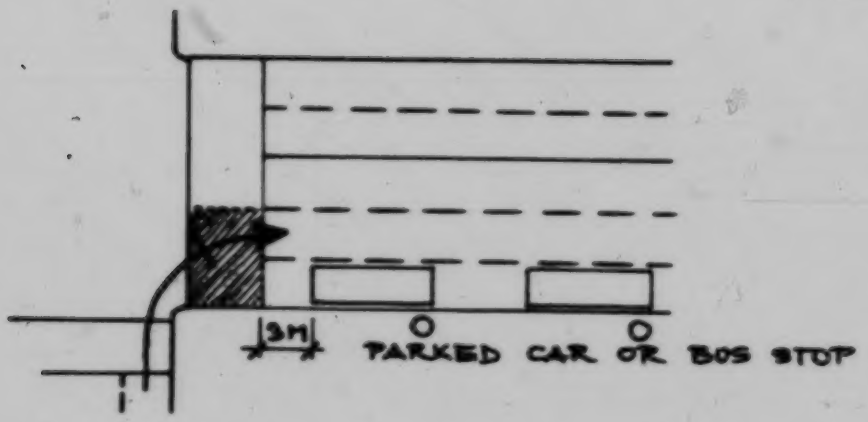


Figure A.7 Obstruction of the first discharge lane

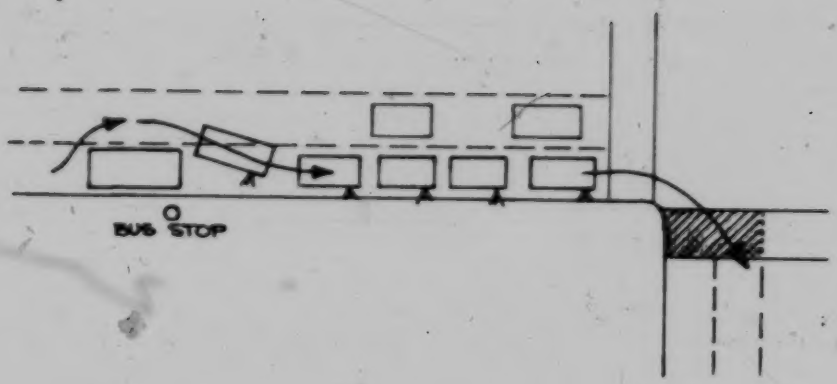


Figure A.8 Obstruction in approach lane with storage

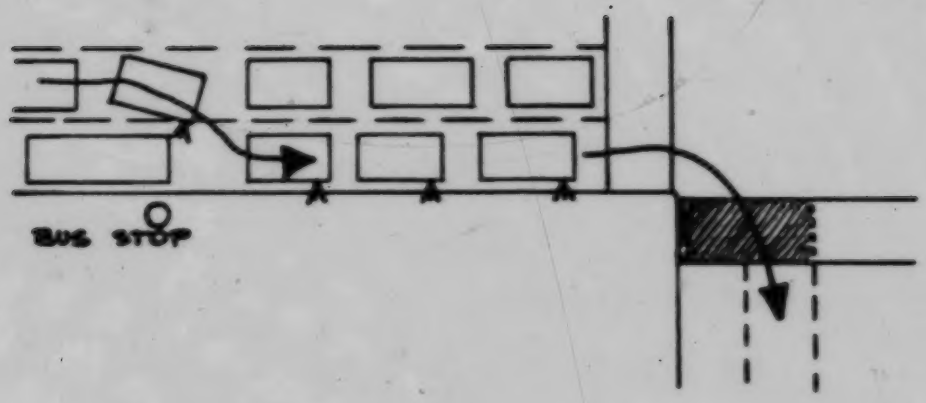


Figure A.9 Pilling the storage from mixed lane

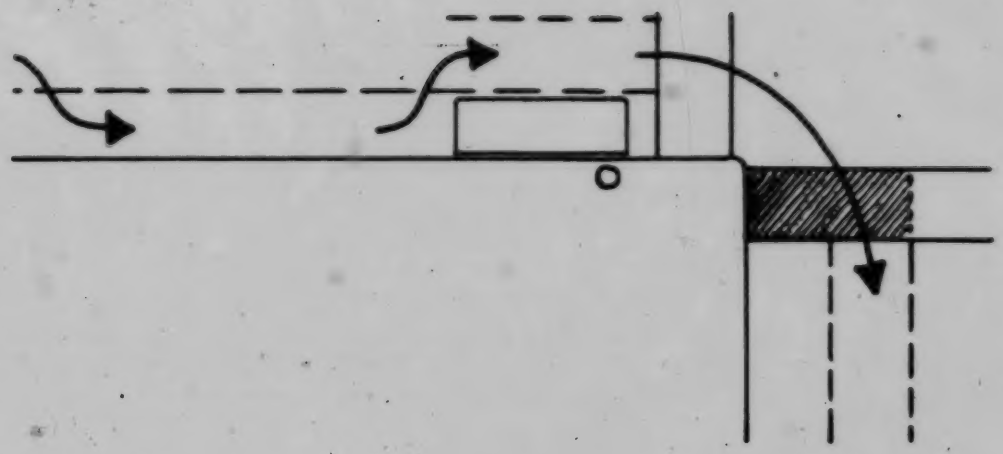


Figure A.10 Obstruction of approach lane without storage

D. The "limited" storage was defined for the formation of the queues consisting of 2-3 vehicles. The right turning flow was difficult to predict, but certain patterns were observed.

a. The locations with high volumes of right turning vehicle flow were found not to be influenced by the obstruction. The mixed traffic in the second lane (with prevailing right turning vehicles) continuously filled the limited storage and right turning flow continued without any interruption.

b. The locations with high volumes of straight through traffic yielded some cycles with interrupted right turn flows (which were excluded from this study). In 50 or more percent of cycles, the flows were not interrupted. This situation was typical for locations with higher pedestrian flows. Vehicles waiting to turn moved at the beginning of the cycle to the stop point. Then they waited for an available gap in the nearside pedestrian flow. Thus the storage space was increased and filled by vehicles from the second lane. Then the "initial" penetration took place and storage space was increased. The farside pedestrian flow created a waiting period as well. During this time the available storage space was filled again by the vehicles from the second lane. The "rest" penetration was not great enough to allow all the

vehicles in the queue to pass the interaction zone and flow was not interrupted (refer to Fig. A.9).

E. Generally, this research was concerned with higher or medium pedestrian flows.

A.3.4 Lanes with right turning and straight through traffic

The right and the straight through lanes were difficult to evaluate. The same geometrical and traffic conditions created different situations. However certain rules were established.

A.3.4.1 First (or first two) vehicles moving straight

A very unusual situation occurred when the first right turning vehicle utilized a gap (sneaking) before the nearside pedestrian platoon entered the interaction zone. The blockade of interaction zone allowed the first two vehicles to move out from the intersection and the creep of the following ones at the stop points. Delaying was not considered. This situation (uninfluenced right turns) would be more pronounced if the storage in front of crosswalk were available.

A.3.4.2 Straight through and right turning vehicles mixed

A. The magnitude of the pedestrian flow played an important role in this case. Straight through (KS) vehicles needed the time to leave and they delayed the right turning flow (K). However, the high volume of pedestrians did not

allow the formation of the gaps to be utilized by more than one right turning vehicle. Thus the real delay was eliminated and right turning flow took place practically uninfluenced. According to the magnitude of turning flow the similarity to the limited queueing space was found.

B. Where the volumes of pedestrians were low the free flow of vehicles was expected. The straight through going vehicles interrupted the free flow. The following assumption were made for evaluation purposes.

a. Occasional vehicles (KS) were evaluated as (K) type vehicles and small differences were neglected.

b. Where more straight through going vehicles were present some adjustment was necessary because they moved faster than turning vehicles. The conversion factor

$$K = KS * \alpha \qquad (26)$$

could be used for evaluation. The coefficient α depends mainly on turning radius, and can be expected to be $\alpha = 1.0$ to 0.7. For this research such cycles were discarded.

c. Extremely high volumes of straight through flows (more than 40% of vehicles) were considered to be a special case which was not covered by this study either.

A.3.5 Opposing flow, motorcycles, semitrailers, turning buses

A. The flow of vehicles from opposing directions turning left did not interact directly with the right turning vehicles. The presence of opposing flow slightly "shocked" the turning drivers and caused a difference in conversion to the pcu.

For some cases the slowing down of the free flow occurred as the turning buses produced a psychological force persuading the right turning driver to stop and yield. This happened only at the end of phase. At that time the stopped vehicle was already at the stop point and finished its maneuver after the bus had left. The cases of total hesitation of the driver to turn right were very rare. The disturbing factor of the opposing flow was not pronounced enough to be mathematically evaluated.

B. Observation of the right turn indicated that 1 pcu is equal to one car, van, or pickup. Motorcycle confirmed its ability to a fast and easy sneak through small gaps. Conversion to 0.5 pcu was found correct. Single unit trucks moved through interaction zone with visible acceleration difficulty. Thus 1.5 pcu was found to be a correct expression.

C. Maneuvering of semitrailers in the downtown areas was found only near the construction sites. The cycles with semitrailers involved were discarded.

D. At seven locations, appearance of buses did not

occur frequently. The straight through going bus could be approximated as 1.75 pcu and converted by coefficient α. For this research the coefficient $\alpha = 1.0$ was used. The right turning buses delayed the vehicle flow as they required a bigger gap to satisfy safety requirements. The equivalent of 2 or 2.25 pcu was found to be adequate.

APPENDIX B - COMPUTER PROGRAMS

Appendix B was created for the presentation of files and programs used during the data manipulation. The system of files is explained together with examples. Programs are presented together with the used logic (flow chart) and examples of data inputs and outputs.

The magnitudes of vehicle flows of different categories (cars, trucks, buses etc.) are obtained as the number of vehicles in 10 second "slices". The same sequential time base was used for pedestrians. The collected data were recorded and presented in a "format" table.

The three methods under evaluation required different arrangements of the input data. The extensive amount of data required computer processing. Computer software was used for sorting, regression analysis, presentation of results, data storage and graphical presentation. Some of the programs were available from system of library (The University of Alberta), some of them had to be written specifically for this project.

B.1. Files

B.1.1 Rough files

The collected data were stored in so-called "rough" (r) files. The passenger car unit equivalent concept was used for the storage of rough data. The arrangement of r-files was the same as in the input "format" table. The number of

passenger car units as well as the number of pedestrians in 10 second "slice" was stored in five columns. Sequence of four rows was assigned to each observed cycle. First row contained nearside pedestrians (P_n), second row farside pedestrians (P_f), third row turning vehicles (K), fourth row straight through (KS) vehicles. First column was reserved for "slice" 0-10 sec, second for 10-20 sec, third for 20-30 sec etc. (Fig. B.1). A computer free format was used for storage of data.

B.1.2 Manipulation files

The files which contained data converted to pcu/h green were called "manipulation" (m) files. They were used as the basic files for any kind of further manipulation. A simple program (TRAN) reversed r-files to m-files where the numbers of vehicles and pedestrians transformed into flow rates. They had the same arrangement as r-files (Fig. B.2).

B.1.3 Working files

Evaluation of the Edmonton method used the data based on a cumulative arrangement. Following the Edmonton concept, the vehicle and pedestrian flows were expressed for slices of: 0-10 sec, 0-20 sec, 0-30 sec, 0-40 sec and 0-50 seconds. A simple computer program (REG) provided the calculation and data arrangements in the working (w) files. The flows of pedestrians and vehicles were expressed in units per hour of green. The files consisted of ten

columns. The first column in a pair represented the independent variable (pedestrian flow rate in ped/h green), the second column presented the dependent variable (vehicle flow rate in pcu/h green). Each line represented one cycle (Fig. B.3).

B.1.4 Combined files

The research required not to evaluate only the sets of observations with the same conditions separately, but also evaluate changes in timing and geometrical conditions and evaluation of all data regardless of influencing factors. Combined (c) files were created for this reason.

B.1.5 Yagar's files

The computer program (COMP) was developed for comparison of all (Toronto, Edmonton, Yagar's) methods. This program used input in m-file format with additional data at the end. The last line presented street width (w), length of intersection zone (w_c), cycle time (c) and green time (G). This kind of file was called Yagar's (y) file (Fig. B.4)

B.1.6 Averaged files

In order to assess the right turning performance on a longer time basis, data from several cycles were averaged. A simple computer program (AVR) used input in w-file format with additional data at the end. The last line presented

```
0-10 sec 'time slice'
10-20 sec
20-30 sec
30-40 sec
40-50 sec

4,7,3,0,0          Nn  }
0,9,1,0,0          Nf  } one cycle
1,1,3.25,2,0       K   }
0,0,1,0,0          KS  }
13,9,1,0,0
0,8,4,0,0          (in numbers of vehicles and pedestrians
0,0,1,3,2           per 10 second slice)
1,0,0,0,0
```

Figure B.1 Example of r-file from "format" table

```
0-10 sec      20-30 sec      40-50 sec 'time slice'
    10-20 sec     30-40 sec

1440.0  2520.0 1080.0   0.0    0.0   Pn  }
   0.0  3240.0  360.0   0.0    0.0   Pf  } one cycle
 360.0   360.0 1170.0 720.0    0.0   K   }
   0.0     0.0  360.0   0.0    0.0   KS  }
4650.0  3240.0  360.0   0.0    0.0
   0.0     0.0  360.0 1080.0 720.0   (in units per
 360.0     0.0    0.0   0.0    0.0    hour of green)
```

Figure B.2 Example of m-file converted from r-file

0-10 sec		0-20 sec		0-30 sec		0-40 sec		0-50 sec	
pedest.	vehicl.	pedest.	vehicl.	pedest.	vehicl.	pedest.	vehicl.	pedest.	vehicl.
1440.0	360.0	3600.0	360.0	2880.0	630.0	2160.0	652.5	1728.0	522.0
4560.0	0.0	5385.0	0.0	4190.0	120.0	3142.5	360.0	2514.0	432.0

Figure B.3 Example of w-file calculated from m-file

information how many cycles to be averaged. This file was called averaged (av) file.

The output file from the program AVR represented so-called working averaged (wa) file. This file was used for regression input.

B.1.7 General notes on files

File identification is shown in Fig. B.5. The alphabetical description was explained in previous paragraphs. The numerical part of file name describes the location by its first two digits. The last digit numbered the data set from a given location. Two digits in the middle described the additional information about the file, for example the time slice.

Some intermediate files had to be occasionally created for organizing during research. They had minor importance and they were used temporarily.

B.2. Programs

This section describes the other programs and routines used during the research. All of these programs use FORTRAN IV language and *FORGTEST compiler. The programs have the following outline:

A. The label describing what the program was used for

B. Description of input was assigned to "read" part

C. Description of the program steps

D. The output file description was assigned to "write"

```
0-10 sec      20-30 sec       40-50 sec 'time slice'
      10-20 sec       30-40 sec
1440.0  2520.0 1080.0    0.0     0.0   Pn  } one cycle
   0.0  3240.0  360.0    0.0     0.0   Pf  }
 360.0   360.0 1170.0  720.0     0.0   K   }
   0.0     0.0  360.0    0.0     0.0   KS  }
4650.0  3240.0  360.0    0.0     0.0
   0.0     0.0  360.0 1080.0   720.0   (in units per
 360.0     0.0    0.0    0.0     0.0    hour of green)
  67.0    21.0   80.0   33.0     0.0
```

green interval(g)(in seconds)
cycle time (C)(in seconds)
width of interaction zone(WC)(in feet)
width of street(W)(in feet)

Figure B.4 Example of Yagar's file

```
03046.w
```

working file
sixth set of measured data
interval 0-40 sec
third location

Figure B.5 Example of file title

part

E. Example of input file

F. Example of output file

B.2.1 Program TRAN

This program transferred the rough data from r-file into the form presented in m-file. The number of vehicles was converted into vehicle flow rates and expressed in pcu/hour of green for each 10 second slice. The same procedure was used for pedestrians. The program output was written into the manipulation m-file. The running procedure was as follows:

```
$RUN *FORTGEST SCARDS=TRAN
$SEB -LOAD# 5=INPUTFILE.R 6=OUTPUTFILE.M
RUN
```

Figure B.6 presents the flow-chart of the program and Figure B.14 its listing.

B.2.2 Program REG

This program arranged the data on a cumulative basis for each 10 second slice. This arrangement was used for regression analysis. The Edmonton method required the use of statistical regression package STPK. The program REG read the input from the manipulation m-files. The output was written into the working w-files. The running procedure

was as follows:

```
$RUN *FORTGTEST SCARDS=REG
$DEB -LOAD# 5=INPUTFILE.M 6=OUTPUTFILE.W
RUN
```

Figure B.7 presents the flow-chart of the program and Figure B.15 its listing.

B.2.3 Program AVR

This program was used for averaging of data. The difference between data averaged in some manner expressed the time bias involved in relationship pedestrians versus vehicles. Averaging was done with previous data sorting (for category analysis) or without sorting (for time bias analysis). The input aw-file had 10 column arrangement. The last line of input file represented number m. It means that each m lines of the data file was averaged. The running procedure was as follows:

```
$RUN *FORTGTEST SCARDS=AVR
$DEB -LOAD# 5=INPUTFILE.AV 6=OUTPUTFILE.WA
RUN
```

Figure B.8 presents the flow-chart of the program and Figure B.16 its listing.

B.2.4 Program COMP

This program was structured for comparison of the three methods available at the present time (Toronto, Edmonton, Yagar's). The program read the input from Yagar's y-file.

A. During the reading the number of stored cycles was counted. The last line of the computer input file yielded information about geometry and timing conditions of the location. All data were separated in the computer memory to distinguish between nearside and farside pedestrians.

B. Depending on the green interval the data describing that part of the cycle were used for the purpose of calculation.

C. For both the Edmonton and Toronto equation vehicle flow in pcu/h green was calculated (as presented in Guide).

D. Yagar's part of the program calculated "initial" and "rest" gaps and the probability of passing vehicles. The theoretical number of the right turns was calculated and presented as rate of flow (in pcu/h green). The pedestrian flow rate was also expressed in pedestrians/h green.

E. The next part of the program compared the calculated results with the measured vehicle flow. The measured flow was taken as a "true" value and expressed as 100%. The positive or negative percentage difference were calculated for each measured cycle.

F. The total of these differences was averaged for each method and expressed as mathematical difference.

G. The output file had a few alternative options which

were described in the program. It also included the input form for GENPLOT (Ref. 22) routine. The running procedure was as follows:

```
$RUN *FORTGTEST SCARDS=COMP
$DEB -LOAD$ 5=INPUTFILE.Y 6=OUTPUTFILE
RUN
```

Figure B.9 presents the flow-chart of the program and Figure B.17 its listing.

B.2.5 Program POIN

This was the supporting program which selected the required pairs of the data from the w-files and formed the input for routine GENPLOT (Ref. 22). The sets of data were presented for each time slice and divided from each other by optional asterisks. The lines with asterisks were later edited with additional information in the GENPLOT routine. The running procedure was as follows:

```
$RUN *FORTGTEST SCARDS=POIN
$DEB -LOAD# 5=INPUTFILE.W 6=OUTPUTFILE
RUN
```

Figure B.10 presents the flow-chart of the program and Figure B.18 its listing.

B.2.6 Program HIS

This was the supporting program for quantitative evaluation of cycles. The program read input from the r- or m-files. Average numbers of pedestrians and vehicles were calculated for each 10 second slice. The pcu were converted into flows and both sets of data were presented in the output file. The number of data involved in the calculation was also printed for future reference (weighting average). The histograms analyzing the flows in cycle were constructed according to this program output. The running procedure was as follows:

```
$RUN *FORTGTEST SCARDS=HIS
$DEB -LOAD$ 5=INPUTFILE.M 6=OUTPUTFILE
RUN
```

Figure B.11 presents the flow-chart of the program and Figure B.19 its listing.

B.2.7 Program CAL

This simple program for creating datapoints was developed to satisfy input requirements of GENPLOT method. Each curve given by the equation from regression analysis was converted into an arbitrary number of data points. The running procedure was as follows:

```
$RUN *FORTGTEST SCARDS=CAL
```

```
$RUN -LOAD# 6=-OUTPUTFILE
RUN
```

Figure B.12 presents the flow-chart of the program and Figure B.20 its listing.

B.2.8 Program FIN

This program applied adjustment factors to the observed data. The input file used the maximum flows detected by using the HIS program. The output file presented original data, flow adjustment factor, adjusted data, radius adjustment factor and final adjusted data. The running procedure was as follows:

```
$RUN *FORTGTEST SCARDS=FIN
$DEB -LOAD# 5=INPUTFILE.W 6=OUTPUTFILE
RUN
```

Before running, the part of the program containing the information about the intersection must be individually changed. This part contains the width of the street (w), radius (r), green time (G) and pedestrian walk time (g). Figure B.13 presents the flow-chart and Figure B.21 its listing.

B.3. Routines - The University of Alberta library

B.3.1 Program *STPK-POLYREG & HIST

Polynominal regression plots (POLYREG) were used for statistical evaluation of data. The ST-Package was developed by the Department of Statistics at the University of Waterloo and is commonly used at the University of Alberta (Ref. 21).

The program used interactive graphical techniques. The advantage of this method was the possibility of using the graphical terminal. The major disadvantage was the limitaiton of regression analysis up to 200 input data points. The program used w-files input, plotted the data points and drew the regression lines (1st to 4th degree) together with (95%) confidence limits. The graph was accompanied by statistical data, such as mean square error (MSE), R-square, F-test (with its P-value) and correlation coefficient as well as with predicted equation. The method based on minimizing squares of errors was used for evaluation of trends. The running procedure was as follows:

```
$RUN *STPK
- interacting with method
POLYREG OR HIST
- interacting with method to get regression figure
<RETURN>
- interacting with method to get hard copy if needed
```

ST-Package contains also subroutines providing the analysis of data normally distributed. The routine presents data in a histogram, provides normal distribution with mean and variance as well as the chi density function.

STPK routines should preferably be used with a graphical terminal but usage of deckwriter is also possible. Examples of the final products are given in Figures C.11, C.12 and C.52.

B.3.2 Program GENPLOT

Multiple plotting program was used for the presentation of results in an easily understandable form. This software was developed by T. Casey and R. Lasby, Department of Civil Engineering, The University of Alberta. The program read the input from two files, PLOTTITLE and PLOTINFO. Input file PLOTTITLE described the location, main title, titles of the plot axis, legend and its position. Inputfile PLOTINFO stored the information about axis scale, axis length and working and axis originals. The number and the type of data points (POIN or COMP outputfile) and the type of plotted lines is also in this file.

The program procedure was stored in the file PLOTMAIN. The advantage of this method was the utilization of the graphical terminal. The final product was adjusted by moving, adding, rotating of the text etc. The major disadvantage was the necessity of creating the sequence of data points for each curve. The data points were created by

using program CAL. The final product (graph) was converted to the output file (-pdf) and printed on the graphical printer.

The running procedure was stored in the file PLOTSO. The procedure for obtaining the hard copy was stored in the file HCOPY. The running procedure used calling the source:

```
$SO PLOTSO
```

for hard copy release

```
$SO HCOPY
```

Listing of input files GENPLOT, PLOTINFO and PLOTTITLE is presented in Figures B.22, B.23 and B.24. Listing of sources is presented in Figures B.25 and B.26. Example of graph (final product) as can be seen in Figure 5.1.

B.3.3 Program SORTIT

The randomly collected data in the sets were generally used without sorting. In some cases of evaluation the sorted data was required. The routine SORTIT was used for this manipulation. The method is stored in The University of Alberta library. The input file was in the form of any numerical matrix, in our case the working w-file. The sorting parameter was given by the computer field expressed in the starting (N1) and the ending (N2) columns. The

sorting had accending order (A). The running procedure was as follows:

```
$RUN *SORT PAR=SORT=CH,A,NI,N2 5=INPUTFILE 6=OUTPUTFILE
```

B.4. Flow-charts

B.5. Listing of Programs

Figure B.6 Flow-chart - program "TRAN"

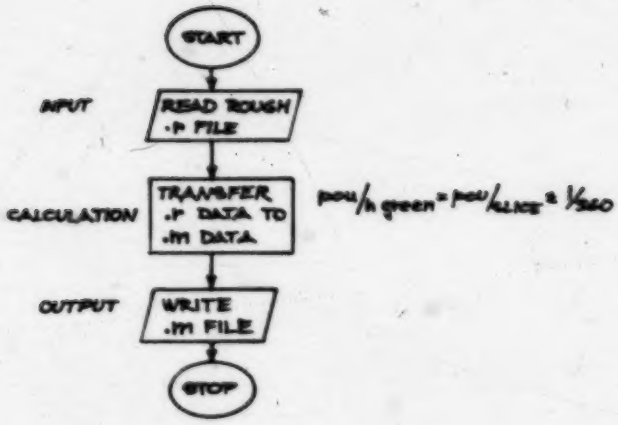


Figure B.7 Flow-chart - program "REG"

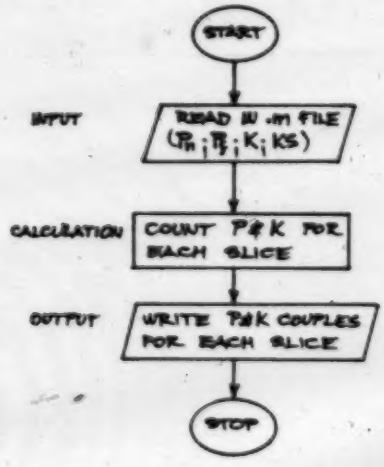


Figure B.8 Flow-chart - program "AVR"

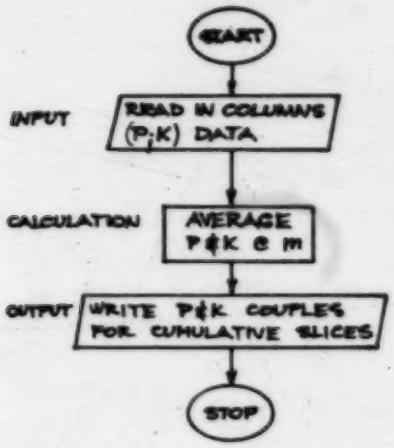
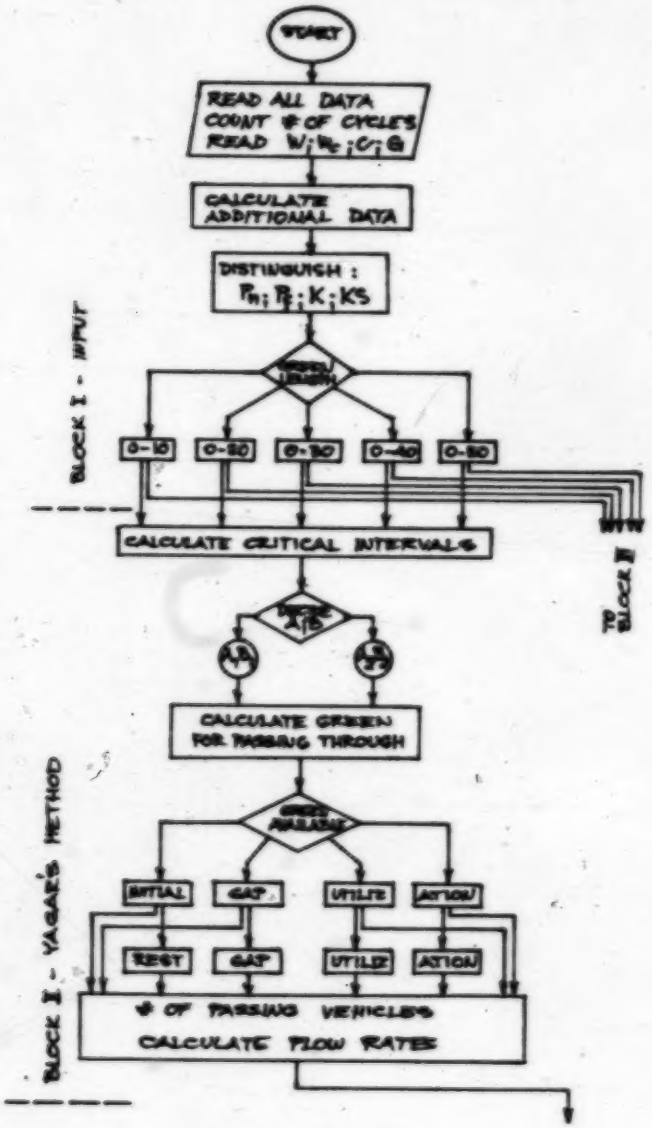


Figure B.9 Flow-chart - program "COMP"



Flow-chart: program "COMP" continued

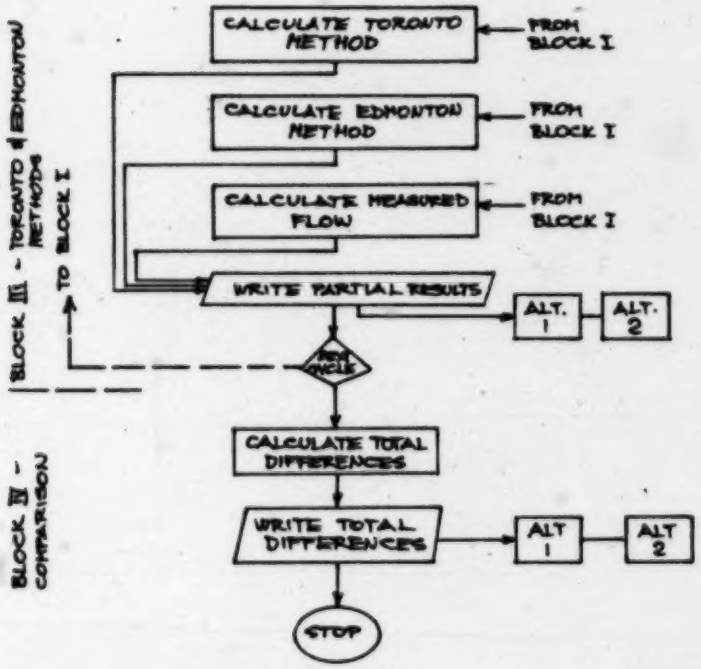


Figure B.10 Flow-chart - program "POIN"

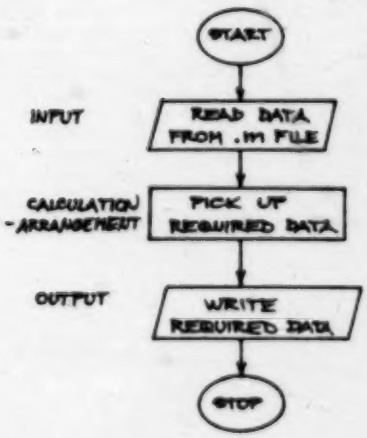


Figure B.11 Flow-chart - program "HIS"

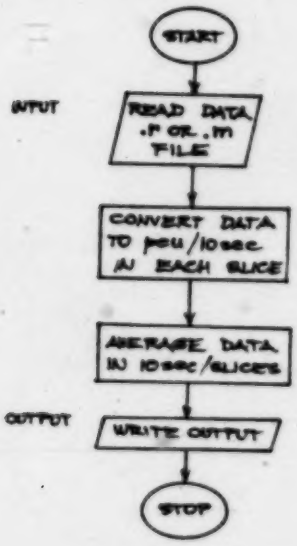


Figure B.12 Flow-chart - program "CAL"

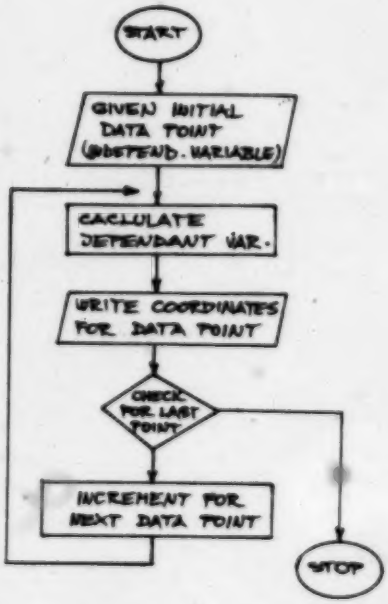


Figure B.13 Flow-chart - program "FIN"

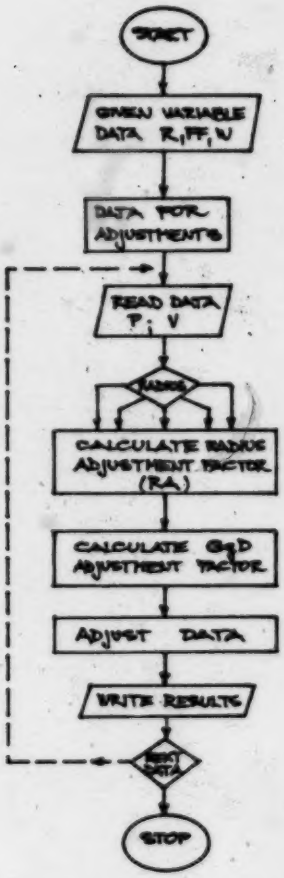


Figure B.14 TRAN

```
      C  .....................TRAN.....................................
      C  THIS PROGRAM READS ROWS OF 10s TIME CHOPS FROM ROUGH 'R'
      C  FILE AND TRANSFERS THEM INTO VALUES PER HOUR OF GREEN
      C  ____________________________________________________________
      C  .........! UP TO 900 LINES IN INPUT FILE ! SEE DIMENSION
            DIMENSION A(900),B(900),C(900),D(900),E(900)
      C  ..............COUNT # OF LINES AND READ DATA
               I=0
             1 I=I+1
               READ(5,2,END=3)A(I),B(I),C(I),D(I),E(I)
             2 FORMAT(5F8.1)
               GO TO 1
             3 CONTINUE
               I=I-1
               N=I
      C  ..............CONVERT DATA
               DO 20 I=1,N
               A(I)=A(I)*360.0
               B(I)=B(I)*360.0
               C(I)=C(I)*360.0
               D(I)=D(I)*360.0
               E(I)=E(I)*360.0
            20 CONTINUE
      C  ..............WRITE DATA
               DO 90 I=1,N
            90 WRITE(6,95)A(I),B(I),C(I),D(I),E(I)
            95 FORMAT(5F11.1)
               STOP
      C  ..............................
      C  EXAMPLE OF INPUT FILE(.r)  FREE FORMAT
      C
      C  5.,0.,0.,0.,0.,
      C  4.,1.,1.,0.,0.,
      C  1.,1.,1.,0.,0.,
      C  0.,0.,0.,0.,0.,
      C  1.,0.,0.,0.,0.,
      C  0.,4.,1.,0.,0.,
      C  1.75,0.,1.,1.,0.,
      C  0.,0.,0.,0.,0.,
      C  ..............................
      C  EXAMPLE OF OUTPUT FILE(.m)   FORMAT(5F11.1)
      C
      C     1800.0        0.0         0.0         0.0         0.0
      C     1440.0      360.0       360.0         0.0         0.0
      C      360.0      360.0       360.0         0.0         0.0
      C        0.0        0.0         0.0         0.0         0.0
      C      360.0        0.0         0.0         0.0         0.0
      C        0.0     1440.0       360.0         0.0         0.0
      C      630.0        0.0       360.0       360.0         0.0
      C        0.0        0.0         0.0         0.0         0.0
      C
            END
End of file
```

Figure B.15 REG

```
      C ..........................REG.....................................
      C THIS PROGRAM READS ROWS OF 10e TIME CHOPS & COL ARRANGEMENT OF
      C PN,PF,C,Cstraight,AND WRITES PED.+ VEHICLES IN 10e ARRANGEMENT
      C                        CARS AND PED./HOUR OF GREEN
      C .........! UP TO 900 LINES IN INPUT FILE ! SEE DIMENSION
            DIMENSION A(900),B(900),C(900),D(900),E(900)
            DIMENSION P1(500),C1(500),P2(500),C2(500),P3(500),C3(500),
           &C4(500),P4(500),C5(500),P5(500)
      C ............COUNT # OF LINES AND READ DATA
            I=0
          1 I=I+1
            READ(5,2,END=3)A(I),B(I),C(I),D(I),E(I)
          2 FORMAT(5F11.1)
            GO TO 1
          3 CONTINUE
            I=I-2
            N=I
      C ..............READ CYCLE TIME (IC)FREE FORMAT
            J=I+1
            IC=A(J)
      C ....CALCULATE # OF PEDESTRIANS AND VEHICLES FOR 10e ARR.
            M=N/4
            DO 20 I=1,M
            J=4*I-3
            K=4*I-2
            L=4*I-1
            P1(I)=A(J)+A(K)
            C1(I)=A(L)
            P2(I)=(A(J)+A(K)+B(J)+B(K))/2.
            C2(I)=(A(L)+B(L))/2.
            P3(I)=(A(J)+A(K)+B(J)+B(K)+C(J)+C(K))/3.
            C3(I)=(A(L)+B(L)+C(L))/3.
            P4(I)=(A(J)+A(K)+B(J)+B(K)+C(J)+C(K)+D(J)+D(K))/4.
            C4(I)=(A(L)+B(L)+C(L)+D(L))/4.
            P5(I)=(A(J)+A(K)+B(J)+B(K)+C(J)+C(K)+D(J)+D(K)+E(J)+E(K))/5.
            C5(I)=(A(L)+B(L)+C(L)+D(L)+E(L))/5.
           20 CONTINUE
      C ....WRITE RESULTS IN 10e ARRANGEMENT(cumulative) 0-10,0-20,etc.
            DO 40 I=1,M
           40 WRITE(6,50)P1(I),C1(I),P2(I),C2(I),P3(I),C3(I),P4(I),C4(I),
             &P5(I),C5(I)
           50 FORMAT(10F8.1)
            STOP
      C ..........................................
      C EXAMPLE OF INPUT FILE(.m) FORMAT(5F11.1),
      C     !!MODIFIED!!          LAST LINE IS IC
      C
      C    1800.0       0.0        0.0        0.0        0.0
      C    1440.0     360.0      360.0        0.0        0.0
      C     360.0     360.0      360.0        0.0        0.0
      C       0.0       0.0        0.0        0.0        0.0
      C     360.0       0.0        0.0        0.0        0.0
      C       0.0    1440.0      360.0        0.0        0.0
      C     630.0       0.0      360.0      360.0        0.0
      C       0.0       0.0        0.0        0.0        0.0
      C      80.0
      C ..........................................
      C EXAMPLE OF OUTPUT FILE(.w)  FORMAT(10F8.1)
      C
      C 3240.0   360.0  1800.0   360.0  1320.0   360.0   960.0   270.0   792.0   216.0
      C  360.0   630.0   900.0   315.0   720.0   330.0   840.0   337.5   432.0   270.0
      C
            END
End of file
```

Figure B.16 AVR

```
      C  ..............................AVR.................................
      C  THIS PROGRAM READS THE DATA IN COLUMNS AVERAGES THEM @ K
      C  AND WRITES THEM IN COLUMNS
      C  ____________________________________________________________________
      C  ..........! UP TO 900 LINES IN INPUT FILE ! SEE DIMENSION
            DIMENSION A(900),B(900),C(900),D(900),E(900),F(900),G(900),
           &H(900),X(900),Y(900),SUMC(450),SUMD(450),SUME(450),SUMF(450)
            DIMENSION SUMG(450),SUMH(450),SUMX(450),SUMY(450),SUMA(450),
           &SUMB(450)
      C  .............COUNT # OF LINES AND READ DATA
            I=0
          1 I=I+1
      C  FORMAT AND DATA TO BE READ DEPENDS ON INPUT FILE
            READ(5,2,END=3)A(I),B(I),C(I),D(I),E(I),F(I),G(I),H(I),X(I),
           &Y(I)
          2 FORMAT(10F8.1)
            GO TO 1
          3 CONTINUE
            I=I-2
            N=I
      C  .......READ NUMBER OF DATA TO BE AVERAGED 'K'
            J=I+1
            K=A(J)
      C  ....AVERAGE THE DATA ACCORDING TO K
            I2=I/K
            DO 20 I=1,I2
            SUMA(I)=0
            SUMB(I)=0
            SUMC(I)=0
            SUMD(I)=0
            SUME(I)=0
            SUMF(I)=0
            SUMG(I)=0
            SUMH(I)=0
            SUMX(I)=0
            SUMY(I)=0
            L=I*K
            I1=I*K-(K-1)
            DO 18 J=I1,L
            SUMA(I)=SUMA(I)+A(J)
            SUMB(I)=SUMB(I)+B(J)
            SUMC(I)=SUMC(I)+C(J)
            SUMD(I)=SUMD(I)+D(J)
            SUME(I)=SUME(I)+E(J)
            SUMF(I)=SUMF(I)+F(J)
            SUMG(I)=SUMG(I)+G(J)
            SUMH(I)=SUMH(I)+H(J)
            SUMX(I)=SUMX(I)+X(J)
            SUMY(I)=SUMY(I)+Y(J)
         18 CONTINUE
            SUMA(I)=SUMA(I)/K
            SUMB(I)=SUMB(I)/K
            SUMC(I)=SUMC(I)/K
            SUMD(I)=SUMD(I)/K
            SUME(I)=SUME(I)/K
            SUMF(I)=SUMF(I)/K
            SUMG(I)=SUMG(I)/K
            SUMH(I)=SUMH(I)/K
            SUMX(I)=SUMX(I)/K
            SUMY(I)=SUMY(I)/K
         20 CONTINUE
      C  ....WRITE RESULTS
            DO 30 I=1,I2
         30 WRITE(6,40)SUMA(I),SUMB(I),SUMC(I),SUMD(I),SUME(I),SUMF(I),
           &SUMG(I),SUMH(I),SUMX(I),SUMY(I)
         40 FORMAT(10F8.1)
            STOP
      C  ..................................................
      C  EXAMPLE OF INPUT FILE  FORMAT(10F8.1)
      C     !!MODIFIED!!        LAST LINE IS 'K'
      C
      C 2880.0   360.0  1980.0   360.0  1320.0   360.0   990.0   450.0   792.0   360.0
      C 2880.0   360.0  1980.0   360.0  1320.0   360.0   990.0   450.0   792.0   360.0
      C 1440.0   360.0  1260.0   540.0   840.0   600.0   720.0   630.0   576.0   504.0
      C 2880.0   360.0  1980.0   360.0  1320.0   360.0   990.0   450.0   792.0   360.0
      C 1440.0   360.0  1260.0   540.0   840.0   600.0   720.0   630.0   576.0   504.0
      C 2880.0   360.0  1980.0   360.0  1320.0   360.0   990.0   450.0   792.0   360.0
      C 1440.0   360.0  1260.0   540.0   840.0   600.0   720.0   630.0   576.0   504.0
      C    3.0
      C  ..................................................
      C  EXAMPLE OF OUTPUT FILE  FORMAT(10F8.1)
      C
      C 2400.0   360.0  1740.0   420.0  1160.0   440.0   900.0   510.0   720.0   408.0
      C 2400.0   360.0  1740.0   430.0  1160.0   440.0   900.0   510.0   720.0   408.0
      C
            END
End of File
```

Figure B.17 COMP

```
      C  .....COMP.....
      C     THIS PROGRAM COMPARES EDMONTON,TORONTO AND YAGAR'S METHOD
      C  _____
      C  .....! UP TO 900 LINES IN INPUT FILE ! SEE DIMENSION
            DIMENSION PN(900),PF(900),S(900),T(900),S1(900),H(900),TP(900)
           &,A1(900),A2(900),G1(900),U(900),V(900),T1(900),A(900),YP(900),
           &GR(900),X(900),Y(900),Z(900),CR(900),CS(900),B(900)
            DIMENSION P(900),F(900),FN(900),C1(900),CTR(900),
           &T2(900),Q(900),BR(900),ED(900),EP(900)
      C  ....COUNT # OF LINES AND READ DATA
            I=0
          1 I=I+1
            READ(5,2,END=3)U(I),V(I),X(I),Y(I),Z(I)
          2 FORMAT(5F11.1)
            GO TO 1
          3 CONTINUE
            I=I-2
            N=I
      C  ......READ STREET WITH(W) IN FT..INTERACTION WITH(WC)
      C        IN FT.,CYCLE TIME(C) IN SEC,GREEN TIME IN SEC
            J=I+1
            W=U(J)
            WC=V(J)
            C=X(J)
            G=Y(J)
      C  ......CALCULATE NON GREEN TIME
            R=C-G
      C  ......CALCULATE ALL DATA
            SUT=0
            SUY=0
            SUR=0
            M=N/4
            DO 70 I=1,M
            J=4*I-3
            K=4*I-2
            L=4*I-1
            IF(G.LE.10) GO TO 31
      C     IF(G.EQ.10.0) GO TO 31
            IF(G.LE.20.AND.G.GT.10) GO TO 32
      C     IF(G.EQ.20.0) GO TO 32
            IF(G.LE.30.AND.G.GT.20) GO TO 33
      C     IF(G.EQ.30.0) GO TO 33
            IF(G.LE.40.AND.G.GT.30) GO TO 34
      C     IF(G.EQ.40.0) GO TO 34
            IF(G.GT.40) GO TO 35
      C  0-10S
           31 P(I)=U(J)/360.
              F(I)=U(K)/360.
              CR(I)=U(L)/360.
              GO TO 80
      C  0-20S
           32 P(I)=(U(J)+V(J))/360.
              F(I)=(U(K)+V(K))/360.
              CR(I)=(U(L)+V(L))/360.
              GO TO 80
      C  0-30S
           33 P(I)=(U(J)+V(J)+X(J))/360.
              F(I)=(U(K)+V(K)+X(K))/360.
              CR(I)=(U(L)+V(L)+X(L))/360.
              GO TO 80
      C  0-40S
           34 P(I)=(U(J)+V(J)+X(J)+Y(J))/360.
              F(I)=(U(K)+V(K)+X(K)+Y(K))/360.
              CR(I)=(U(L)+V(L)+X(L)+Y(L))/360.
              GO TO 80
      C  0-50S
           35 P(I)=(U(J)+V(J)+X(J)+Y(J)+Z(J))/360.
              F(I)=(U(K)+V(K)+X(K)+Y(K)+Z(K))/360.
              CR(I)=(U(L)+V(L)+X(L)+Y(L)+Z(L))/360.
              GO TO 80
      C  _____YAGAR_____
           80 PN(I)=P(I)/C
              PF(I)=F(I)/C
      C  ..... PN,PF,IN PED/H....ACTUALLY IN PED./S
      C  ..... CALCULATE CRITICAL INTERVAL
              A1(I)=((R+WC/4.)/(1-.45*PN(I)))-R
              A2(I)=((R+W/4.)/(1-.45*PF(I)))-R
              IF(A1(I).GE.A2(I)) A(I)=A2(I)
              IF(A1(I).GE.A2(I)) B(I)=A1(I)
              IF(A1(I).LT.A2(I)) A(I)=A1(I)
              IF(A1(I).LT.A2(I)) B(I)=A2(I)
      C  .....CALCULATE GREEN
              G1(I)=(W-WC)/4.-(R+WC/4.)/(1.-0.45*PN(I))+R
```

COMP continued

```
C  .....CALCUL OF PROBABILITY OF PASSING VEHICLES INITIAL
      IF(G1(I).LE.5.) C1(I)=0
      IF(G1(I).GT.5.0.AND.G1(I).LE.8.6) C1(I)=EXP(-5.*PN(I)/1.7)
      IF(G1(I).GT.8.6.AND.G1(I).LE.12.2) C1(I)=EXP(-5.*PN(I)/1.7)+
     &EXP(-8.6*PN(I)/1.7)
      IF(G1(I).GT.12.2.AND.G1(I).LE.15.8) C1(I)=EXP(-5.*PN(I)/1.7)+
     &EXP(-8.6*PN(I)/1.7)+EXP(-12.2*PN(I)/1.7)
      IF(G1(I).GT.15.8.AND.G1(I).LE.19.4) C1(I)=EXP(-5.*PN(I)/1.7)+
     &EXP(-8.6*PN(I)/1.7)+EXP(-12.2*PN(I)/1.7)+EXP(-15.8*PN(I)/1.7)
      IF(G1(I).GT.19.4.AND.G1(I).LE.23.0) C1(I)=EXP(-5.*PN(I)/1.7)+
     &EXP(-8.6*PN(I)/1.7)+EXP(-12.2*PN(I)/1.7)+EXP(-15.8*PN(I)/1.7)
     &+EXP(-19.4*PN(I)/1.7)
C  .....CALC OF PROBAB OF PASSING VEH  REST
      Q(I)=(PN(I)+PF(I))/1.7
      GR(I)=G-B(I)
      IF(GR(I).LE.5.) CTR(I)=1.
      IF(GR(I).GT.5.0.AND.GR(I).LE.8.6) CTR(I)=1.+Q(I)
     &*GR(I)*EXP(-5.0*Q(I))
      IF(GR(I).GT.8.6.AND.GR(I).LE.12.2) CTR(I)=1.+Q(I)
     &*GR(I)*(EXP(-5.0*Q(I))+EXP(-8.6*Q(I)))
      IF(GR(I).GT.12.2.AND.GR(I).LE.15.8) CTR(I)=1.+Q(I)
     &*GR(I)*(EXP(-5.0*Q(I))+EXP(-8.6*Q(I)
     &)+EXP(-12.2*Q(I)))
      IF(GR(I).GT.15.8.AND.GR(I).LE.19.4) CTR(I)=1.+Q(I)
     &*GR(I)*(EXP(-5.0*Q(I))+EXP(-8.6*Q(I))
     &+EXP(-12.2*Q(I))+EXP(-15.8*Q(I)))
      IF(GR(I).GT.19.4.AND.GR(I).LE.23.) CTR(I)=1.+Q(I)
     &*GR(I)*(EXP(-5.0*Q(I))+EXP(-8.6*Q(I))+EXP(-12.2*Q(I))+
     &EXP(-15.8*Q(I))+EXP(-19.4*Q(I)))
      BR(I)=EXP(-5.0*Q(I))+EXP(-8.6*Q(I))+EXP(-12.2*Q(I))+
     &EXP(-15.8*Q(I))+EXP(-19.4*Q(I))+EXP(-23.0*Q(I))
      IF(GR(I).GT.23.AND.GR(I).LE.26.6) CTR(I)=1.+Q(I)*GR(I)*BR(I)
C  .....NUMBER OF PASSING CARS/CYCLE
      T(I)=C1(I)+CTR(I)
C  .....RATE OF VEHICLE FLOW / HOUR
      T1(I)=T(I)*3600/C
C  .....RATE OF VEHICLE FLOW / HOUR OF GREEN
      T2(I)=T(I)*3600/G
C  .....CALCULATE PEDESTRIAN FLOW
      S(I)=(PN(I)+PF(I))*3600
C                              IN PED/HOUR...FROM YAGAR FORM.
      S1(I)=(P(I)+F(I))/G*3600
C                               IN PED/H OF GREEN
C  .....REAL RATE OF VEH / HOUR
      H(I)=CR(I)*3600/C
C                               IN VEH/HOUR
      FN(I)=CR(I)/G*3600
C                               IN VEH/H OF GREEN
C                 TORONTO
C  .....CALCULATE VEHICLE FLOW / H OF GREEN
      CS(I)=1073-S1(I)*0.21
C                 EDMONTON
C  .....CALCULATE VEHICLE FLOW / H OF GREEN
      ED(I)=100+1450/EXP(0.001*S1(I))
C  .....CALCULATE DIFFERENCE IN %
      TP(I)=(CS(I)-FN(I))/FN(I)*100
      YP(I)=(T2(I)-FN(I))/FN(I)*100
      EP(I)=(ED(I)-FN(I))/FN(I)*100
      SUT=SUT+CS(I)
      SUY=SUY+T2(I)
      SUE=SUE+ED(I)
      SUR=SUR+FN(I)
   70 CONTINUE
C  .....WRITE RESULTS
C                ADD 1 :  COMPARISON
C  .....COMPUTED TOTALS
      WRITE(6,80)
   80 FORMAT('   PED/G  REAL V/G  TOR V/G   ED V/G  YAG V/G',
     &'   YAG V/C  INIT    REST    TOR%  ED% YAG%')
      DO 95 I=1,M
   95 WRITE(6,96)S1(I),FN(I),CS(I),ED(I),T2(I),T(I),
     &C1(I),CTR(I),TP(I),EP(I),YP(I)
   96 FORMAT(5F8.1,3X,3F7.2,3X,3F5.0)
C  .....AVERAGE DIFERENCES IN VALUES OF OBSERVED CYCLE
      FUT=SUT/M
      FUY=SUY/M
      FUE=SUE/M
      FUR=SUR/M
      DT=-1*(FUR-FUT)/FUR*100
      DY=-1*(FUR-FUY)/FUR*100
      DE=-1*(FUR-FUE)/FUR*100
   97 WRITE(6,98)
   98 FORMAT('-----------------------------------')
```

COMP continued - input output files

```
      100 WRITE(6,101)
      101 FORMAT(' AVERAGE REAL',8X,'AVER.TOR.',3X,'AVER.YAG.',3X,
     &'DIF.TOR.',3X,'DIF.EDM.',3X,'DIF.YAG')
      102 WRITE(6,105)FUR,FUT,FUY,DT,DE,DY
      105 FORMAT(3X,F8.2,4X,F8.2,3X,F8.2,3X,F5.0,6X,F5.0,6X,F5.0,/)
C                        ADD 2:DATA POINTS
C                       .OPTIONAL PART FOR PLOT
C                        (** - NUMBER OF POINTS=N)
      200 WRITE(6,201)
      201 FORMAT('**,1,1,1,0.1,')
          DO 220 I=1,N
          WRITE(6,202)S(I),FN(I)
      202 FORMAT(2F8.1)
      220 CONTINUE
          WRITE(6,203)
      203 FORMAT('**,1,1,1,0.1,')
          DO 230 I=1,N
          WRITE(6,204)S(I),T2(I)
      204 FORMAT(2F8.1)
      230 CONTINUE
          STOP
C ..................................................
C  EXAMPLE OF INPUT FILE(.y) FORMAT(5F11.1)
C                                  LAST LINE:W,WC,C,G
C
C 2880.0       0.0       0.0       0.0       0.0
C    0.0    4320.0    3960.0       0.0       0.0
C    0.0       0.0       0.0    1080.0       0.0
C    0.0       0.0       0.0       0.0       0.0
C 2160.0       0.0       0.0       0.0       0.0
C  720.0    5040.0     360.0       0.0       0.0
C    0.0       0.0     180.0     720.0       0.0
C    0.0       0.0       0.0       0.0       0.0
C   67.0      21.0      80.0      33.0       0.0
C
C  EXAMPLE OF OUTPUT FILE
C
C*,1,1,1,0.1,
C 1395.0   327.3
C 1035.0   272.7
C*,1,1,1,0.1,
C 1395.0   163.0
C 1035.0   228.4
C
          END
End of file
```

```
     1      PED/G  REAL V/G  TOR V/G   ED V/G  YAG V/G   YAG V/C  INIT    REST    TOR%  ED%  YAG%
     2     1080.0     600.0    846.2    592.4    424.3      3.54   0.0    3.54    41.   -1.  -29.
     3      720.0     600.0    921.8    805.8    360.6      3.01   0.0    3.01    54.   34.  -40.
     4     1320.0     480.0    785.8    487.3    419.5      3.50   0.0    3.50    66.    2.  -13.
     5      360.0     900.0    997.4   1111.6    278.5      2.32   0.0    2.32    11.   24.  -69.
     6     1320.0     780.0    795.8    487.3    432.2      3.60   0.0    3.60     2.  -38.  -45.
     7     1320.0     480.0    795.8    487.3    399.9      3.33   0.0    3.33    66.    2.  -17.
     8      600.0     660.0    947.0    895.8    446.7      3.72   0.96   2.76    43.   36.  -32.
     9      840.0     480.0    896.6    726.0    376.2      3.14   0.0    3.14    87.   51.  -22.
    10      480.0     960.0    872.2    997.3    312.7      2.61   0.0    2.61     1.    4.  -67.
    11     1680.0     360.0    720.2    370.2    412.3      3.44   0.0    3.44   100.    3.   15.
    12      600.0     480.0    947.0    895.8    446.7      3.72   0.96   2.76    97.   87.   -7.
    13      600.0     600.0    947.0    895.8    339.7      2.83   0.0    2.83    58.   49.  -43.
    14      600.0     600.0    947.0    895.8    446.6      3.72   1.00   2.72    58.   49.  -26.
    15     ------------------------------
    16   AVERAGE REAL      AVER.TOR.    AVER.YAG.    DIF.TOR.    DIF.EDM.    DIF.YAG
    17         613.84          886.91       392.00         44.         21.        -36.
End of file
```

Figure B.18 POIN

```
      C     ..................POIN.....................................
      C     THIS PROGRAM READS 10 COLUMNS OF PEDESTRIANS AND VEHICLES
      C     AND WRITES 2 COLUMN (PEDESTRIANS AND VEHICLES) ARRANGEMENT
      C     FOR PLOTING SUBRUTINE - GENPLOT
      C
      C     ____________________________________________________________
      C     .....! UP TO 200 LINES IN INPUT FILE ! SEE DIMENSION
            DIMENSION A(200),B(200),C(200),D(200),E(200),F(200),
           &G(200),X(200),Y(200),Z(200)
      C     .........COUNT # OF LINES AND READ DATA
            I=0
          1 I=I+1
            READ(5,2,END=3)A(I),B(I),C(I),D(I),E(I),F(I),G(I),
           &X(I),Y(I),Z(I)
          2 FORMAT(10F8.1)
            GO TO 1
          3 CONTINUE
            I=I-1
            N=I
      C     ..........WRITE DATA
            DO 20 I=1,N
         20 WRITE(6,30)A(I),B(I)
         30 FORMAT(2F8.1)
      C     **** OPTION WRITEN BETWEEN EACH DATA SET, IF 'Cs'
      C       ARE ERRASED-40,50,80,90,120,130,160,170,200,210
      C  40 WRITE(6,50)
      C  50 FORMAT('****')
            DO 60 I=1,N
         60 WRITE(6,70)C(I),D(I)
         70 FORMAT(2F8.1)
         80 WRITE(6,90)
         90 FORMAT('****')
            DO 100 I=1,N
        100 WRITE(6,110)E(I),F(I)
        110 FORMAT(2F8.1)
      C 120 WRITE(6,130)
      C 130 FORMAT('****')
            DO 140 I=1,N
        140 WRITE(6,150)G(I),X(I)
        150 FORMAT(2F8.1)
        160 WRITE(6,170)
        170 FORMAT('****')
            DO 180 I=1,N
        180 WRITE(6,190)Y(I),Z(I)
        190 FORMAT(2F8.1)
      C 200 WRITE(6,210)
      C 210 FORMAT('****')
            STOP
      C     ..........................................
      C     EXAMPLE OF INPUT DATA FILE(.w) FORMAT(10F8.1)
      C
      C 2880.0   360.0  1980.0   360.0  1320.0   360.0   990.0   450.0   792.0   360.0
      C 1440.0   360.0  1260.0   540.0   840.0   600.0   720.0   630.0   576.0   504.0
      C     ..........................................
      C     EXAMPLE OF OUTPUT FILE  FORMAT(2F8.1)
      C
      C 2880.0   360.0
      C 1440.0   360.0
      C 1980.0   360.0
      C 1260.0   540.0
      C 1320.0   360.0
      C           ETC.
      C
            END
End of File
```

Figure B.19 HIS

```
C .....HIS.....
C THIS PROGRAM READS ROWS OF 10s TIME CHOPS FROM ROUGH 'R'
C FILE AND TRANSFERS THEM INTO VALUES PER HOUR OF GREEN.
C OUTPUT IS:AVERAGES OF PEDESTRIANS AND VEHICLES CONVERTED
C PER HOUR OF GREEN,AVERAGES OF PEDESTRIANS AND VEHICLES.
C NUMBER OF OBSERVATIONS        ALL FOR 10s SLICES
C
C .....! UP TO 900 LINES IN INPUT FILE ! SEE DIMENSION
      DIMENSION A(900),B(900),C(900),D(900),E(900),A1(900),
     &A2(900),B1(900),B2(900),C1(900),C2(900),D1(900),D2(900)
     &,E1(900),E2(900)
C .....COUNT # OF LINES AND READ DATA
      I=0
    1 I=I+1
      READ(5,2,END=3)A(I),B(I),C(I),D(I),E(I)
    2 FORMAT(5F8.1)
C ...FOR 'M' FILES FORMAT(5F11.1)....
C   2 FORMAT(5F11.1)
      GO TO 1
    3 CONTINUE
      I=I-1
      N=I
C .....CONVERT DATA
      M=N/4
      DO 20 I=1,M
      J=4*I-3
      K=4*I-2
      L=4*I-1
      A1(I)=A(J)+A(K)
      A2(I)=A(L)
      B1(I)=B(J)+B(K)
      B2(I)=B(L)
      C1(I)=C(J)+C(K)
      C2(I)=C(L)
      D1(I)=D(J)+D(K)
      D2(I)=D(L)
      E1(I)=E(J)+E(K)
      E2(I)=E(L)
   20 CONTINUE
C .....AVERAGE DATA
      DO 30 I=1,M
      AA=AA+A1(I)
      AB=AB+A2(I)
      BA=BA+B1(I)
      BB=BB+B2(I)
      CA=CA+C1(I)
      CB=CB+C2(I)
      DA=DA+D1(I)
      DB=DB+D2(I)
      EA=EA+E1(I)
      EB=EB+E2(I)
   30 CONTINUE
      AA=AA/M
      AB=AB/M
      BA=BA/M
      BB=BB/M
      CA=CA/M
      CB=CB/M
      DA=DA/M
      DB=DB/M
```

HIS continued

```
      EA=EA/M
      EB=EB/M
C     .............CONVERT DATA
      A11=AA*360
      A21=AB*360
      B11=BA*360
      B21=BB*360
      C11=CA*360
      C21=CB*360
      D11=DA*360
      D21=DB*360
      E11=EA*360
      E21=EB*360
C     .............WRITE DATA
   50 WRITE(6,90)
   51 WRITE(6,96)AA,BA,CA,DA,EA
   52 WRITE(6,91)
   53 WRITE(6,96)AB,BB,CB,DB,EB
   54 WRITE(6,92)
   55 WRITE(6,96)A11,B11,C11,D11,E11
   56 WRITE(6,93)
   57 WRITE(6,96)A21,B21,C21,D21,E21
   58 WRITE(6,94)
   59 WRITE(6,95)M
   90 FORMAT('.........PEDESTRIANS.........')
   91 FORMAT('.........VEHICLES...........')
   92 FORMAT('.........PEDESTRIANS/HOUR OF GREEN.....')
   93 FORMAT('.........VEHICLES/HOUR OF GREEN.........')
   94 FORMAT('.........NUMBER OF OBSERVATIONS.........')
   95 FORMAT(1I3)
   96 FORMAT(5F8.2)
      STOP
C  ..............................
C  EXAMPLE OF INPUT FILE(.r) FREE FORMAT
C                    OR FILE(.m) FORMAT(5F11.1)
C  5.,0.,0.,0.,0.,
C  4.,1.,1.,0.,0.,
C  1.,1.,1.,0.,0.,
C  0.,0.,0.,0.,0.,
C  1.,0.,0.,0.,0.,
C  0.,4.,1.,0.,0.,
C  1.75,0.,1.,1.,0.,
C  0.,0.,0.,0.,0.,
C  ..............................
C  EXAMPLE OF OUTPUT FILE        FORMAT(5F8.2)
C
C.......PEDESTRIANS.......
C   5.00    2.50    1.00   -0.00   -0.00
C.......VEHICLES..........
C   1.38    0.50    1.00    0.50   -0.00
C.......PEDESTRIANS/HOUR OF GREEN.....
C1800.00  900.00  360.00   -0.00   -0.00
C.......VEHICLES/HOUR OF GREEN.........
C 495.00  180.00  360.00  180.00   -0.00
C.......NUMBER OF OBSERVATIONS.........
C 2
C
      END
End of file
```

Figure B.20 CAL

```
1          ..........CAL.............
2                REAL X,Y
3                X=100
4              2 Y=180+1370/EXP(0.0007*X)
5                WRITE(6,201)X,Y
6            201 FORMAT(F6.1,F8.2)
7                IF(X.GE.5000)GO TO 1
8                X=X+100
9                GO TO 2
10             1 STOP
11               END
12
```

Figure B.21 FIN

```
1    C ..........................FIN.....................................
2    C       PROGRAM 'FIN' APPLIES FINAL ADJUSTMENTS
3    C       TO THE OBSERVED DATA. PROGRAM USES RADIUS
4    C       AND FREE FLOW FACTORS. FINAL OUPUT
5    C       REPRESENTS THE GENERAL LOCATION.
6    C ---------------------------------------------------------------------
7    C ..............! UP TO 900 DATA !.............
8          DIMENSION P(900),V(900),RA(900),FA(900),T(900)
9          DIMENSION F(900),S(900),S2(900),S3(900)
10   C .........READ RADIUS,FREE FLOW,STREET WIDTH
11   C
12   C      INPUT FOR EACH LOCATION: R,FF,W
13           R=13.0
14           FF=10
15           W=22.0
16           Z=2000.0
17           Z1=2000.0
18           B=280.0
19           C=0.8
20           D=2.2
21           S1=180
22   C
23   C ..........COUNT # OF LINES AND READ MAX. OBSERVED
24   C            PEDESTRIAN & VEHICLE FLOW RATE
25   C
26           I=0
27         1 I=I+1
28           READ(5,2,END=3)P(I),V(I)
29         2 FORMAT(2F11.1)
30           GO TO 1
31         3 CONTINUE
32           I=I-1
33           N=I
34   C ............ADJUST DATA
35           DO 8 I=1,N
36   C .........RADIUS ADJ.
37           IF(P(I).GT.3700) GO TO 27
38        10 IF(R.GT.4.5.AND.R.LE.5.5) Y=0.6
39           IF(R.GT.5.5.AND.R.LE.6.5) Y=0.625
40           IF(R.GT.6.5.AND.R.LE.7.5) Y=0.66
41           IF(R.GT.7.5.AND.R.LE.8.5) Y=0.72
42           IF(R.GT.8.5.AND.R.LE.9.5) Y=0.74
43           IF(R.GT.9.5.AND.R.LE.10.5) Y=0.81
44           IF(R.GT.10.5.AND.R.LE.11.5) Y=0.87
45           IF(R.GT.11.5.AND.R.LE.12.5) Y=0.92
46           IF(R.GT.12.5.AND.R.LE.13.5) Y=0.97
47           IF(R.GT.13.5.AND.R.LE.14.5) Y=0.99
48           IF(R.GT.14.5) Y=1.0
49        20 RA(I)=Y+P(I)*(1-Y)/Z
50           GO TO 28
51        27 RA(I)=1.0
52   C .........FREE FLOW ADJ.
53        28 VE=W/FF
54           IF(P(I).GT.Z1) GO TO 35
55           Y1=(VE-C)*B/(D-C)
56           FA(I)=Y1-Y1*P(I)/Z1
57           GO TO 36
58        35 FA(I)=0.0
59        36 T(I)=V(I)+FA(I)
60           F(I)=T(I)/RA(I)
```

FIN continued

```
      C  .........WRITE RESULTS
        40 WRITE(6,80)P(I),V(I),T(I),F(I),RA(I),FA(I)
        80 FORMAT(6F8.1)
         5 CONTINUE
       100 WRITE(6,110)R,FF,W
       110 FORMAT(3F8.1)
           STOP
      C  ..................................
      C  EXAMPLE OF INPUT FILE   FORMAT(2F8.1)
      C
      C      360.0      900.0
      C      480.0      960.0
      C      600.0      480.0
      C      720.0      600.0
      C      840.0      480.0
      C     1080.0      600.0
      C     1320.0      480.0
      C     1680.0      360.0
      C  ..................................
      C  EXAMPLE OF OUTPUT FILE  FORMAT(6F8.1)
      C
      C  360.0   900.0  1128.6  1158.1     1.0   229.6
      C  480.0   960.0  1172.8  1200.2     1.0   212.8
      C  600.0   480.0   676.0   690.5     1.0   196.0
      C  720.0   600.0   778.2   794.9     1.0   178.2
      C  840.0   480.0   642.4   653.8     1.0   162.4
      C 1080.0   600.0   728.8   738.0     1.0   128.8
      C 1320.0   480.0   575.2   581.1     1.0    95.2
      C 1680.0   360.0   404.8   406.8     1.0    44.8
      C   13.0    10.0    22.0
      C
           END
End of File
```

Figure B.22 GENPLOT

```
13
14          .....PLOTMAIN.....
15           DATA XCDEF,YCDEF, KADEF, KBDEF, KCDEF, SIZDEF, ICPEN /5.0,9.0,
16          &1, 1, 2, 0.15, 1/
17           NCMAX=32
18           CALL GENPLT(NCMAX,XCDEF,YCDEF,KADEF,KBDEF,KCDEF,SIZDEF,ICPEN)
19           STOP
20           END
21
```

Figure B.23 PLOTINFO

```
22
23          .....PLOTINFO.....
24          0.500,16,
25          0.200,8,
26          13,1,1,1,0.1
27                360.0       900.0
28                480.0       960.0
29                600.0       600.0
30                600.0       660.0
31                600.0       480.0
32                600.0       600.0
33                720.0       600.0
34                840.0       480.0
35               1080.0       600.0
36               1320.0       780.0
37               1320.0       480.0
38               1320.0       480.0
39
```

Figure B.24 PLOTTITLE

```
40
41          .....PLOTTITLE.....
42          EDMONTON LOCATIONS COMBINED - 620 observations
43          PEDESTRIAN FLOW RATE IN PED. PER HOUR OF GREEN
44          VEHICLE FLOW RATE IN VEH. PER HOUR OF GREEN
45          6.5,6,
46          LEGEND
47          1,1,location 6
48          2,1,location 7
49          3,11,Y=100+1450/EXP0.001*X
50          4,10,Y=180+1370/EXP0.0015*X
51          5,13,Y=180+1370/EXP0.001*X
52          6,10,Y=180+1370/EXP0.0007*X
53
```

Figure B.25 PLOTSO

```
54
55          .....PLOTSO.....
56          R *FORTGTEST SCARDS=PLOTMAIN PAR=ID,SOURCE
57          R -LOAD#+GS46:GENPLOT SCARDS=PLOTTITLE 5=PLOTINFO 6=-P 8=-PDF T=75
58
```

Figure B.26 HCOPY

```
59
60          .....HCOPY.....
61          r *calcompq par=file=-pdf  return=civs
End of file
```

APPENDIX C - EVALUATION OF SEVEN LOCATIONS

The purpose of Appendix C is to list the partial results. The process of evaluation, development of Edmonton method and its calibration involved many steps which could not be included in the report due to their quantity. Each chapter or step of research was described in a working paper, which also included all graphs and results of minor importance or work routines. The partial results, graphs or tables are presented in this appendix. Important figures for each paragraph are included in the major part of the thesis.

C.1. Analysis of individual data sets (Location #2)

The graphical presentation of the analysis provided on each data set (Location #2) is presented in Figures C.1 to C.5. Each figure presents an individual cumulative time "slice". The dotted and the dashed lines represent each set according to the different green time. The final polynomial regression lines for the years 1982 and 1983 were superimposed for a better comparison.

C.2 Analysis of averaged data sets (Location #2)

Examples of the regression analysis provided on the averaged data sets are presented in Figures C.6 to C.10. It can be seen the tendency of the illogical results. The result differences between the smoothed and observed data

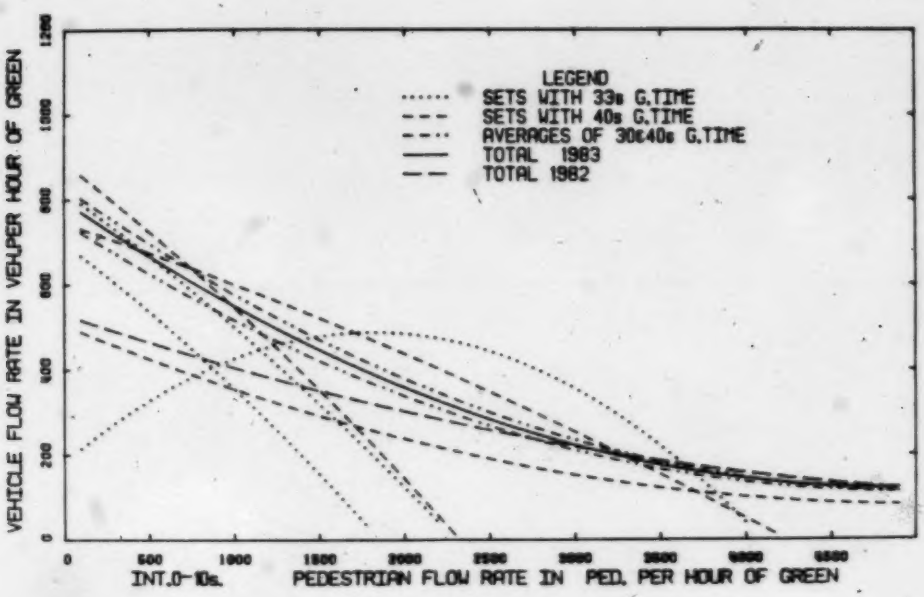


Figure C.1 Location #2 - Regression lines for interval 0-10

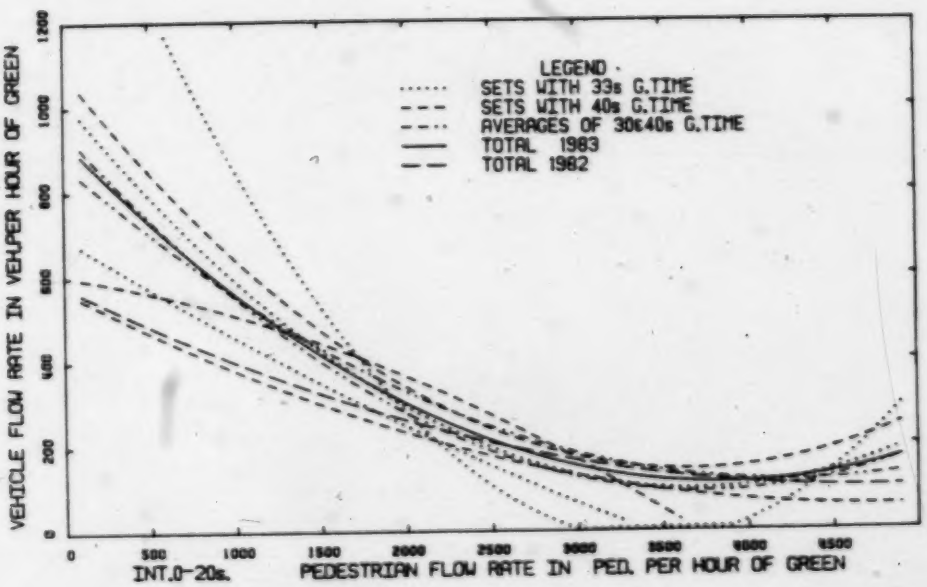


Figure C.2 Location #2 - Regression lines for interval 0-20

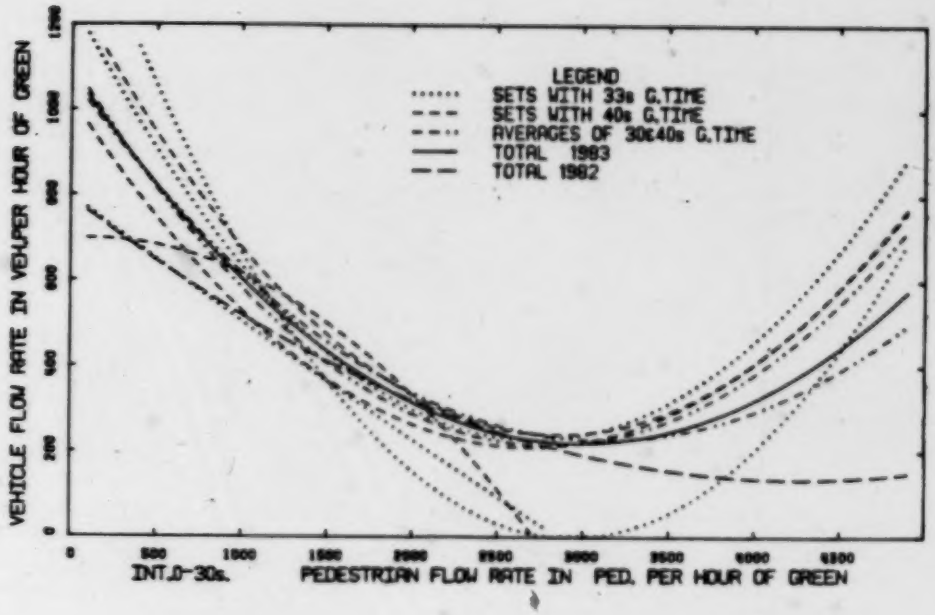


Figure C.3 Location #2 - Regression lines for interval 0-30

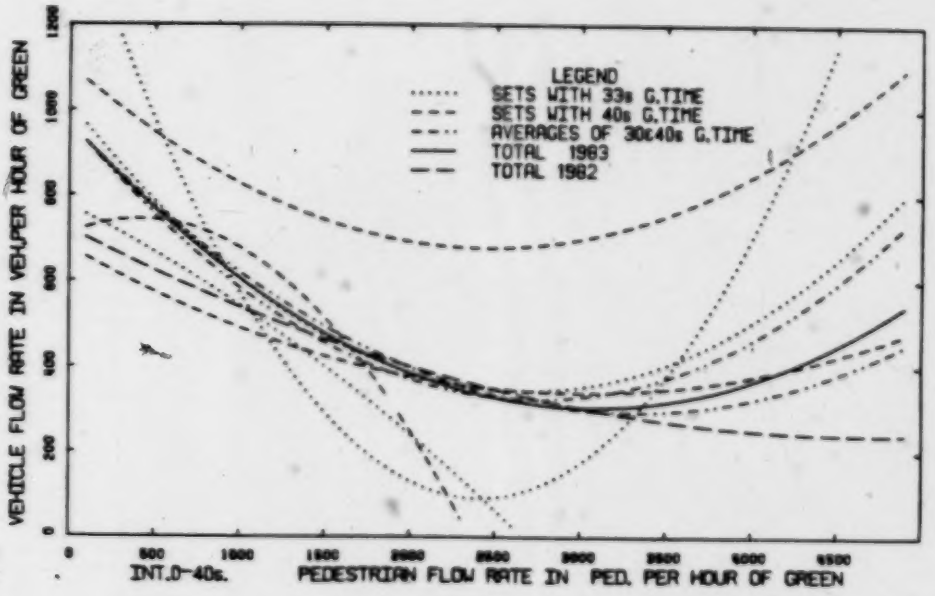


Figure C.4 Location #2 - Regression lines for interval 0-40

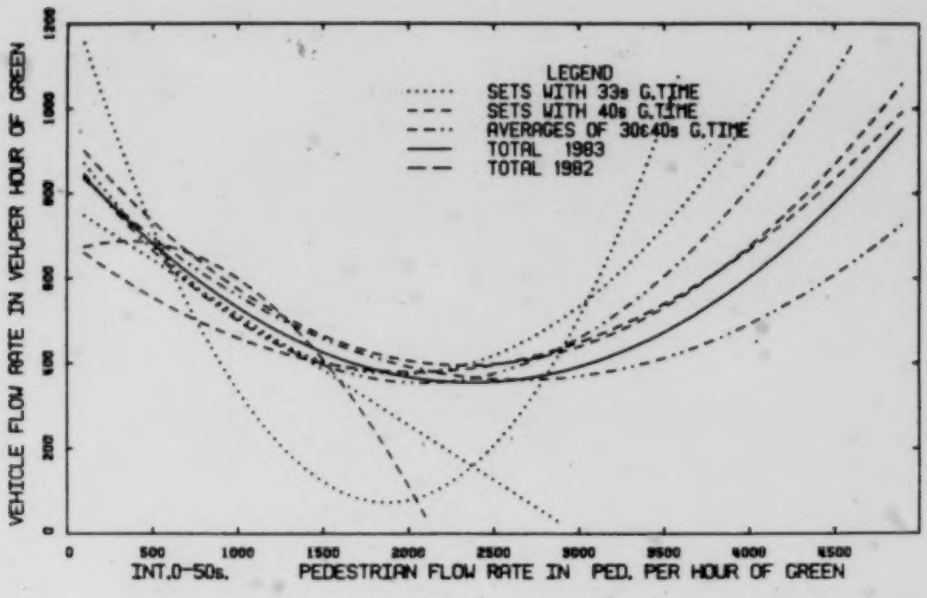


Figure C.5 Location #2 - Regression lines for interval 0-50

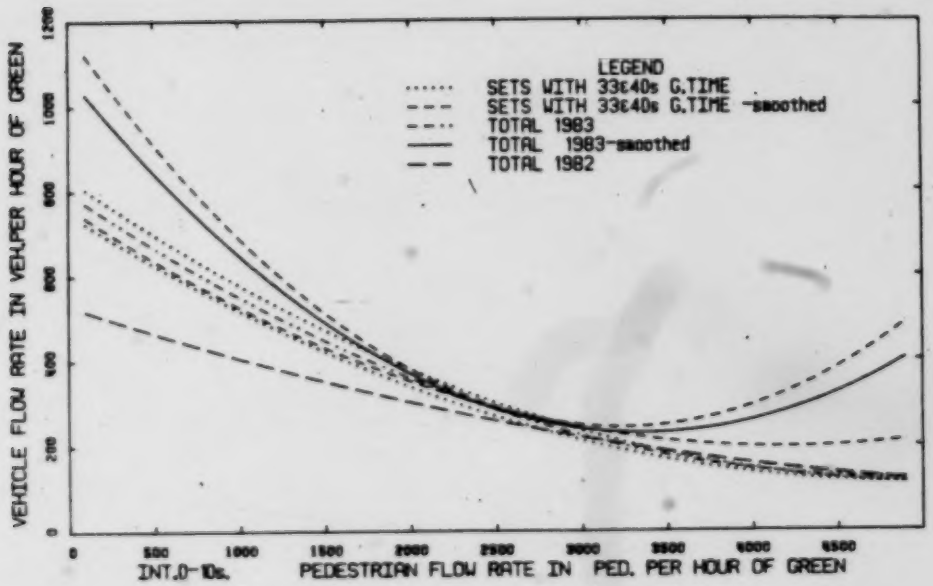


Figure C.6 Location #2 - Interval 0-10 (smoothed data)

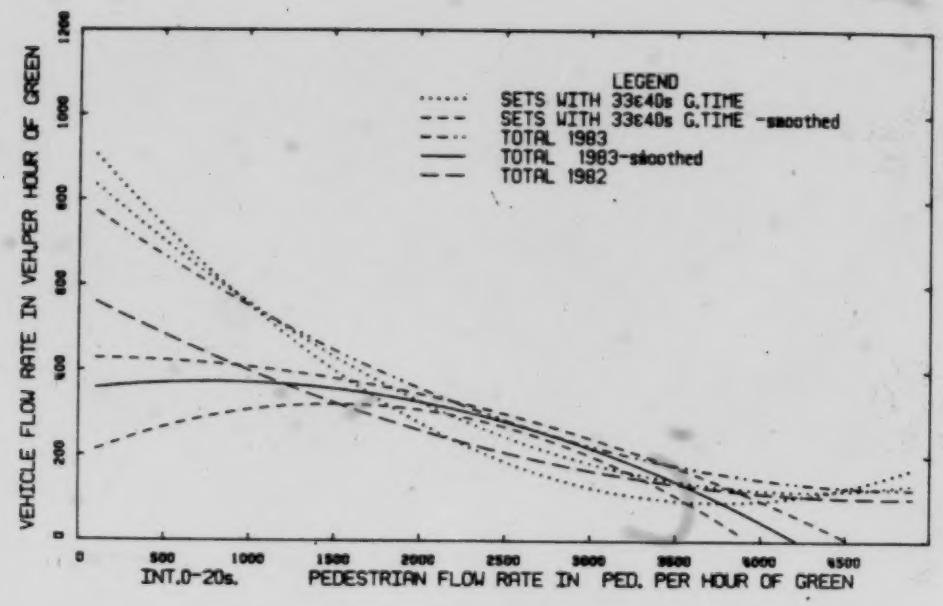


Figure C.7 Location #2 - Interval 0-20 (smoothed data)

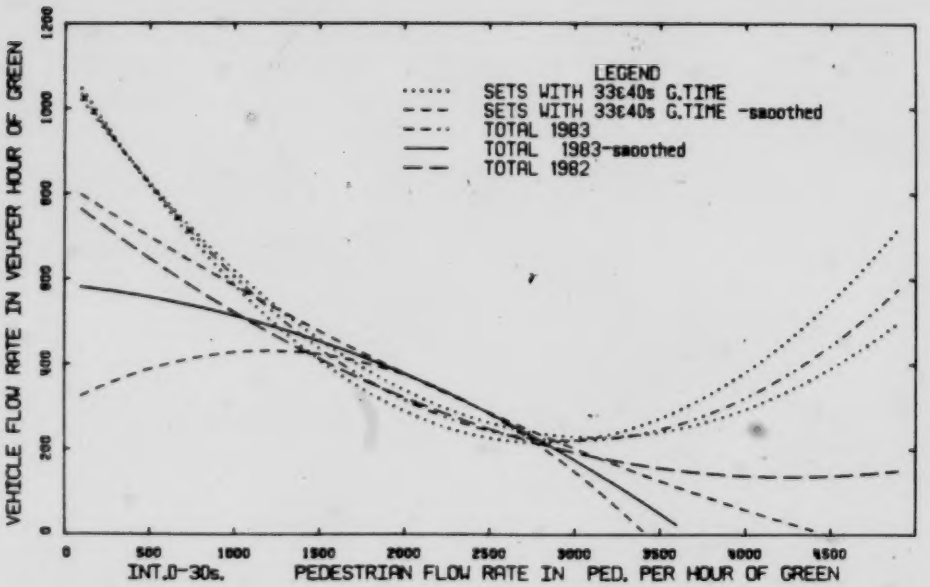


Figure C.8 Location #3 - Interval 0-30 (smoothed data)

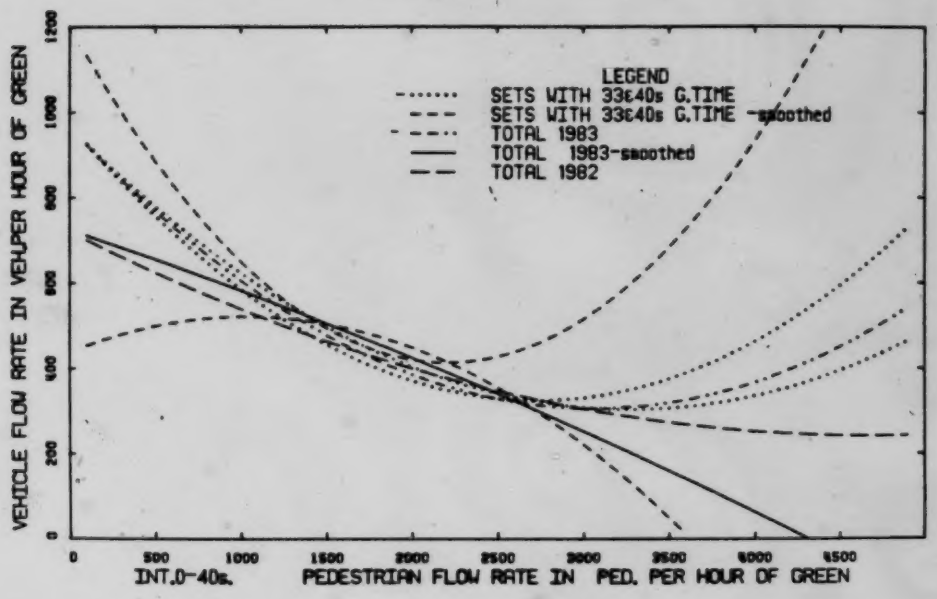


Figure C.9 Location #2 - Interval 0-40 (smoothed data)

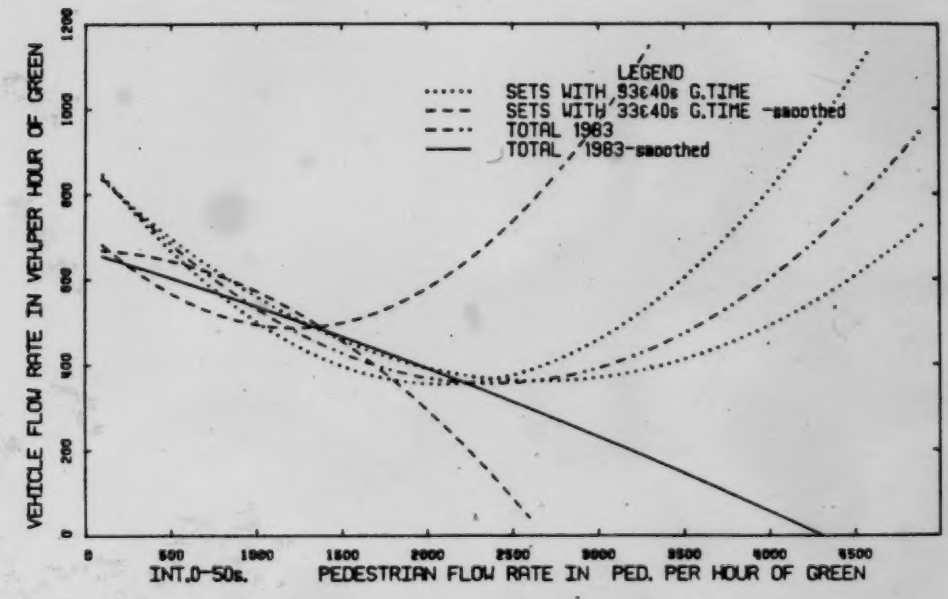

LEGEND
SETS WITH 33&40s G.TIME
SETS WITH 33&40s G.TIME -smoothed
TOTAL 1983
TOTAL 1983-smoothed
VEHICLE FLOW RATE IN VEH/PER HOUR OF GREEN
INT.0-50s.
PEDESTRIAN FLOW RATE IN PED. PER HOUR OF GREEN

Figure C.10 Location #2 - Interval 0-50 (smoothed data)

are evident. The smoothing method was abandoned.

C.3 Analysis of combined data (Location #2)

Regression analyses were performed on all data sets. The statistical package provided the possibility of using first to the fourth degree curves. The output included the regressional equation, sum of R-squares, required confidence band correlation coefficient, mean square error and the graphical expression of regression lines together with the confidence limits.

The regression curves of the third and fourth degree were found to be not adequate. The parabolic curve (second degree) described the searched relationship well. In some cases however the mathematical procedure involved might have hidden or distorted the real trend. This fact must be kept in mind and logical evaluation is necessary. Figures C.11 and C.12 express the output from program STPK-POLYREG. Figures were randomly selected from the analysis made on sets from location #2.

C.4 Polynomial expression

Figure C.13 represents the adjustment for the low point according to the Toronto method. The more correct adjustment, according to the observed facts, was made in Figure 6.13.

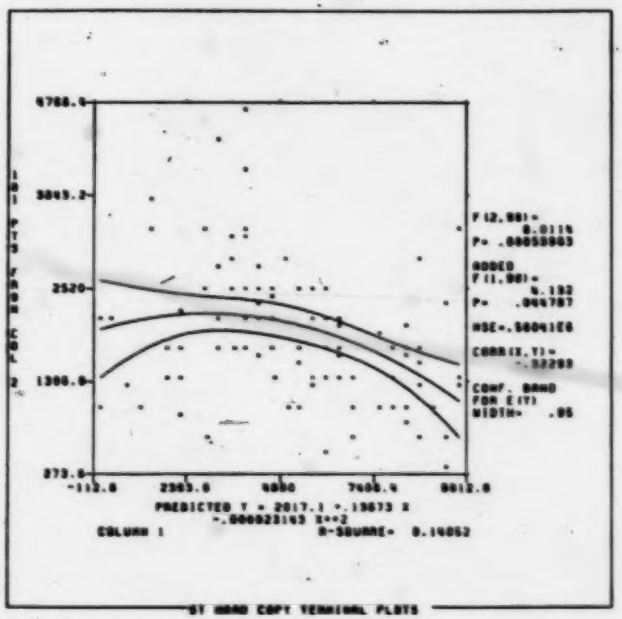


Figure C.11 Example of regression analysis - 2nd degree curve (parabolic)

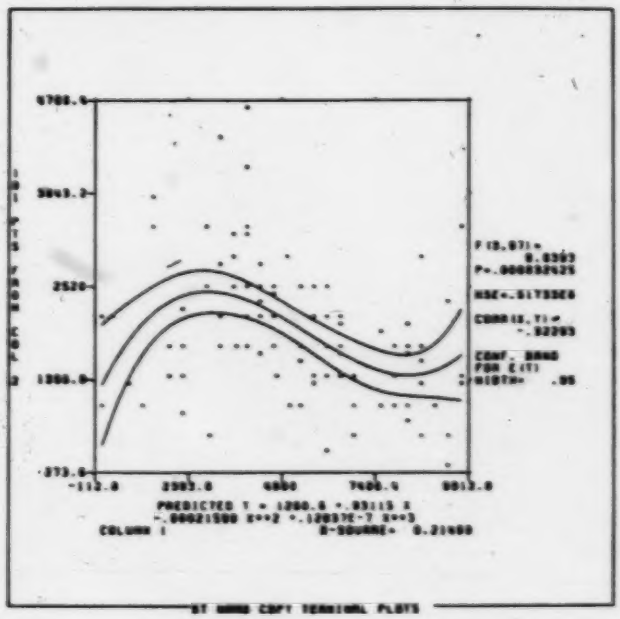


Figure C.12 Example of regression analysis - 3rd degree curve (cubic)

Figure C.14 presents the polynomial curve with superimposed data. It can be seen that the high point also needs adjustment. A strictly mathematical regression did not match the real trend of the observed relationship.

The high point was established as 1550 pcu/h green. The exponential and hyperbolic curves were used in an attempt to find the best fit (Figure C.15).

Figure C.16 represents the data, regression polynomial curve and selected exponential curve. This exponential curve was used in the Canadian Capacity Guide as a tentative solution to the problem.

C.5 Exponential expression

Exponential function in Figure C.17 presents the developed basic formula in the same fashion as in Canadian Capacity Guide. The similarity with opposed left turns can be seen.

C.6 Analysis of Location #1, #3, to #7

Polynomial regressions of second degree applied to locations 1, 3, 4, 5, 6, 7 are presented in Figures C.18 to C.47. The results from regressions of first degree are indicated in Table C-1.

C.7 Data adjusted by radius factor

Data from all seven locations were adjusted by proposed radius adjustment factor and superimposed on basic curve

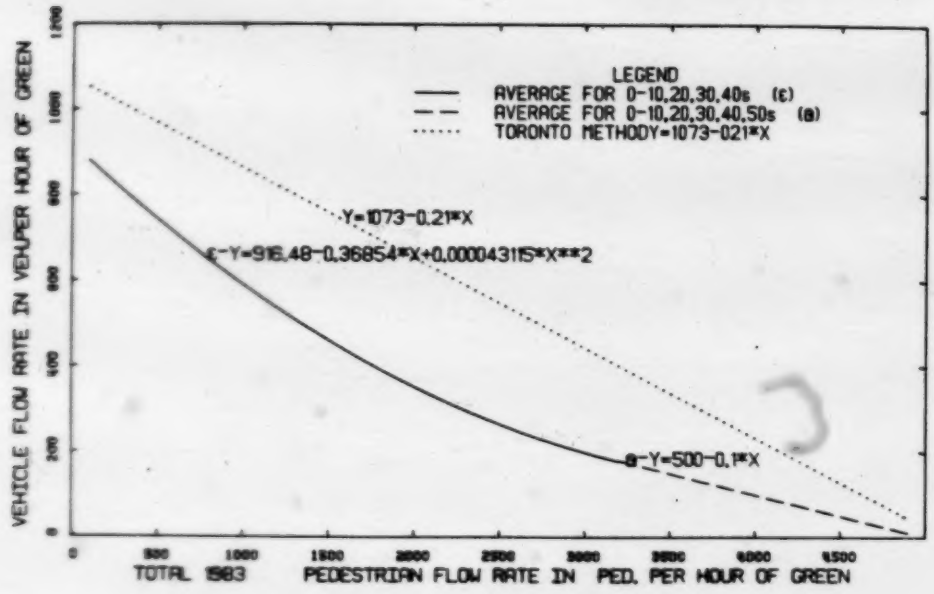


Figure C.13 Adjusted polynomial curve (by Toronto method)

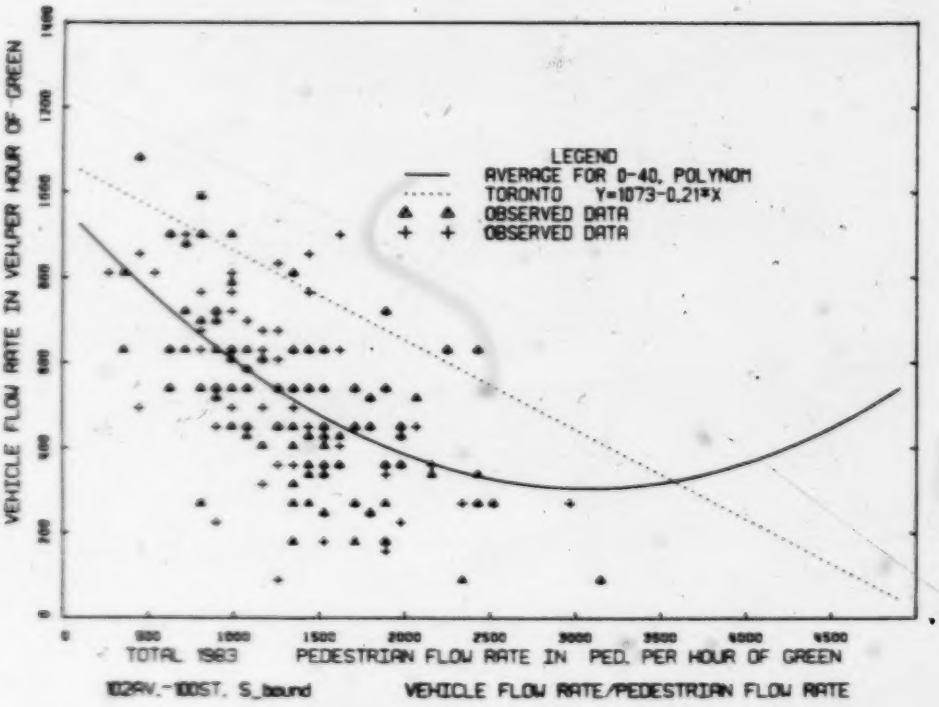


Figure C.14 Polynomial curve and data - used for adjustment

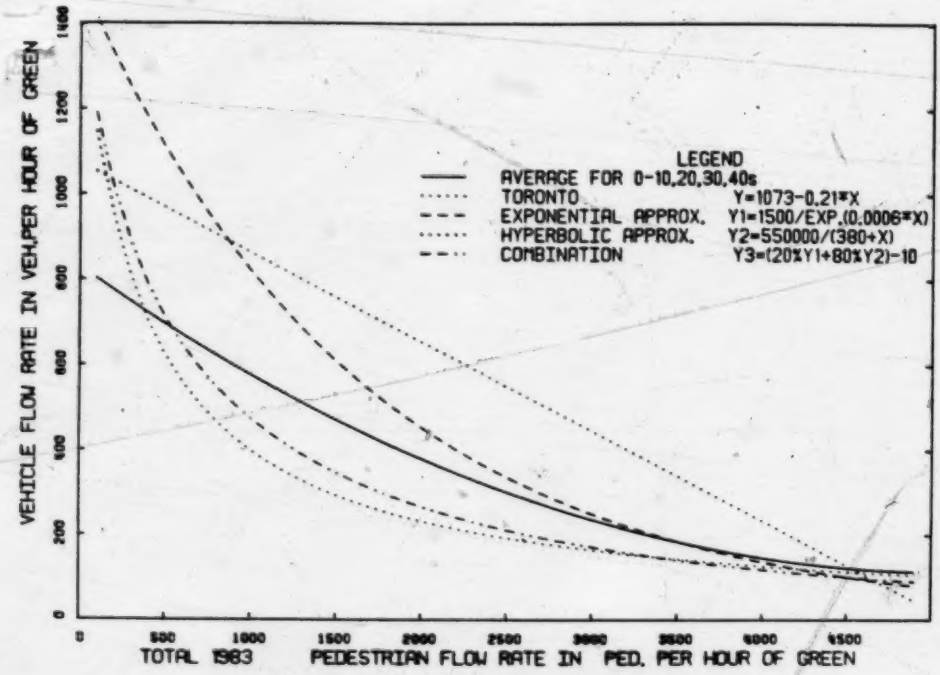

LEGEND
AVERAGE FOR 0-10,20,30,40s
TORONTO Y=1073-0.21*X
EXPONENTIAL APPROX. Y1=1500/EXP.(0.0006*X)
HYPERBOLIC APPROX. Y2=550000/(380+X)
COMBINATION Y3=(20%Y1+80%Y2)-10
VEHICLE FLOW RATE IN VEH/PER HOUR OF GREEN
TOTAL 1983 PEDESTRIAN FLOW RATE IN PED. PER HOUR OF GREEN

Figure C.15 Second degree curves

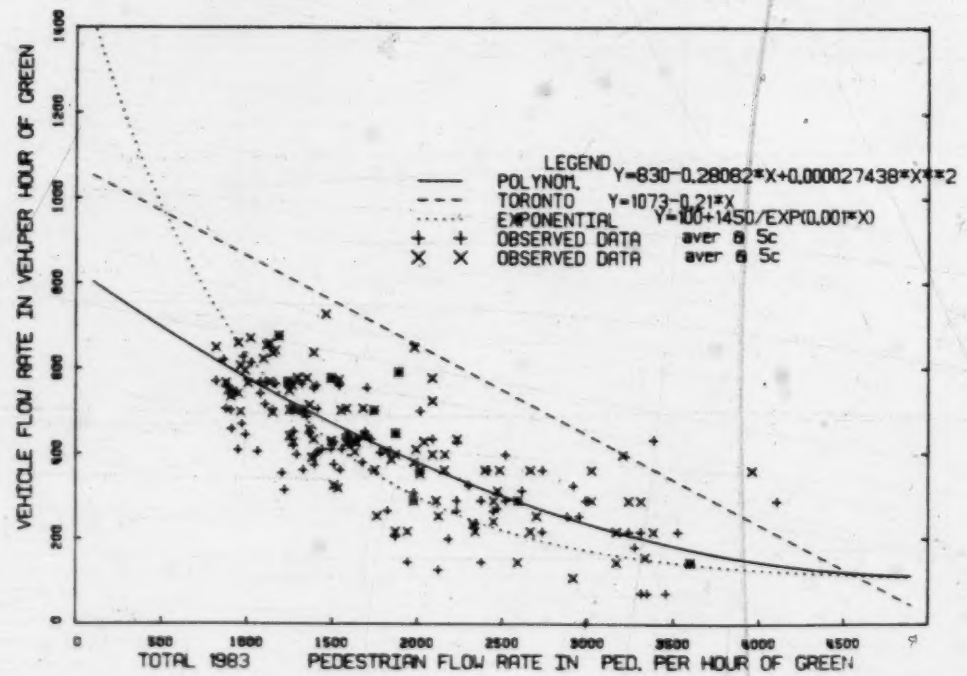

LEGEND
POLYNOM. Y=830-0.28082*X+0.000027438*X**2
TORONTO Y=1073-0.21*X
EXPONENTIAL Y=100+1450/EXP(0.001*X)
OBSERVED DATA aver 8 Sc
OBSERVED DATA aver 8 Sc
VEHICLE FLOW RATE IN VEH/PER HOUR OF GREEN
TOTAL 1983 PEDESTRIAN FLOW RATE IN PED. PER HOUR OF GREEN

Figure C.16 Preliminary exponential fit

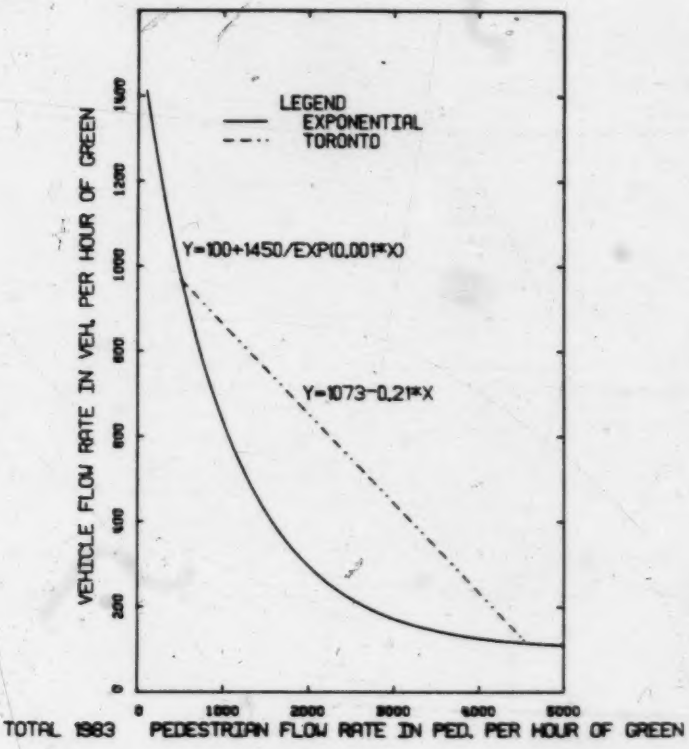


Figure C.17 Exponential curve as presented in Guide

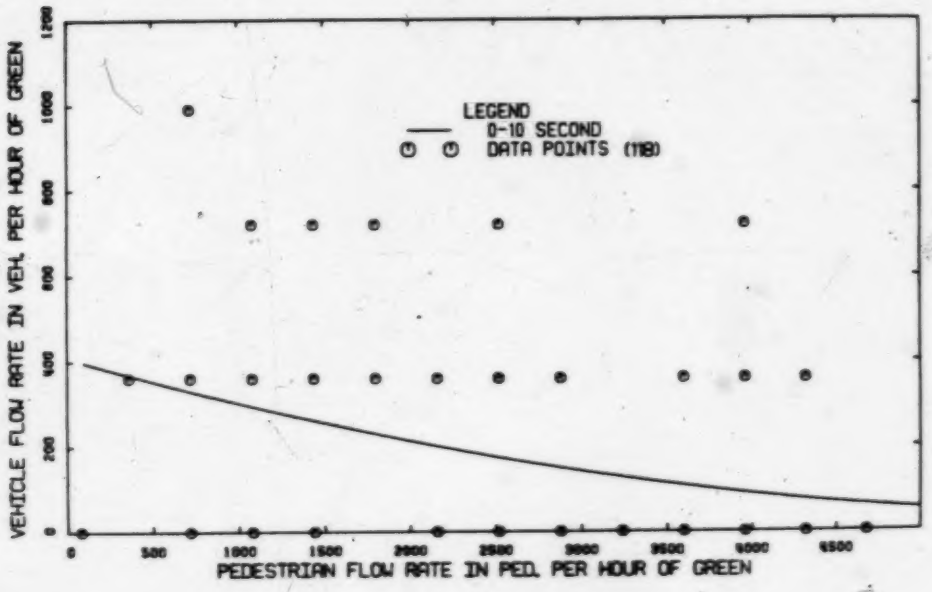


Figure C.18 Location #1; 0-10 sec

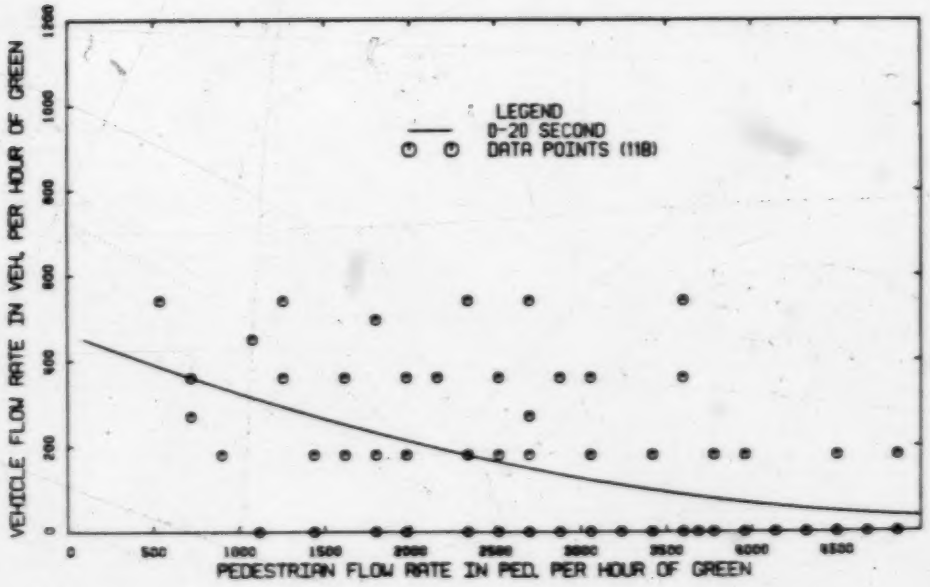


Figure C.19 Location #1; 0-20 sec

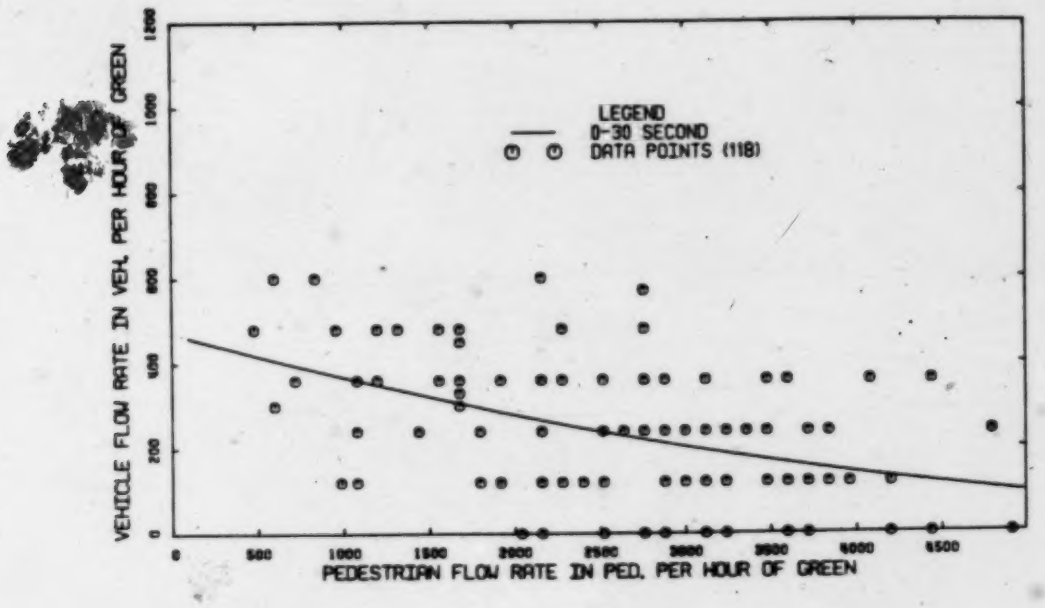


Figure C.20 Location #1; 0-30 sec

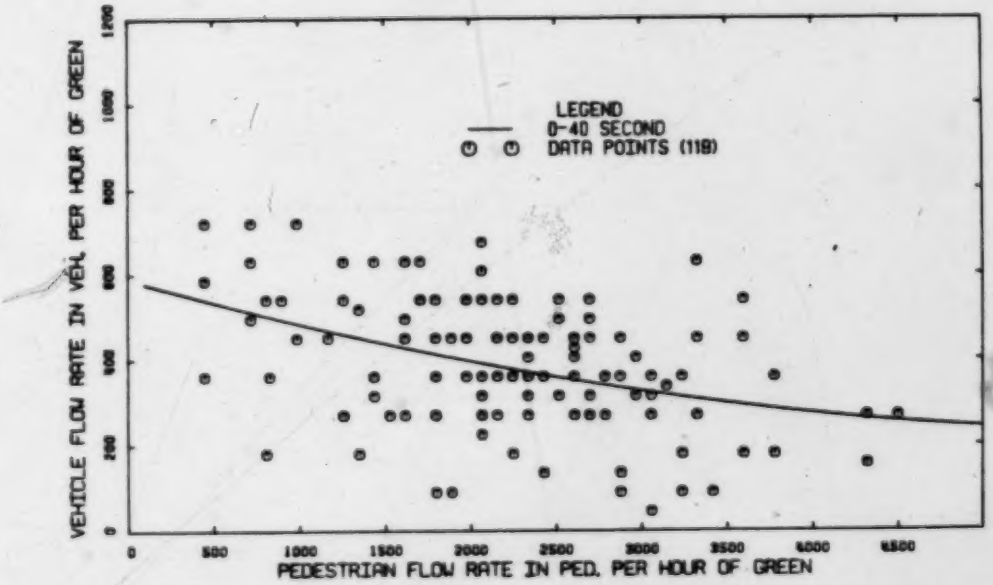


Figure C.21 Location #1; 0-40 sec

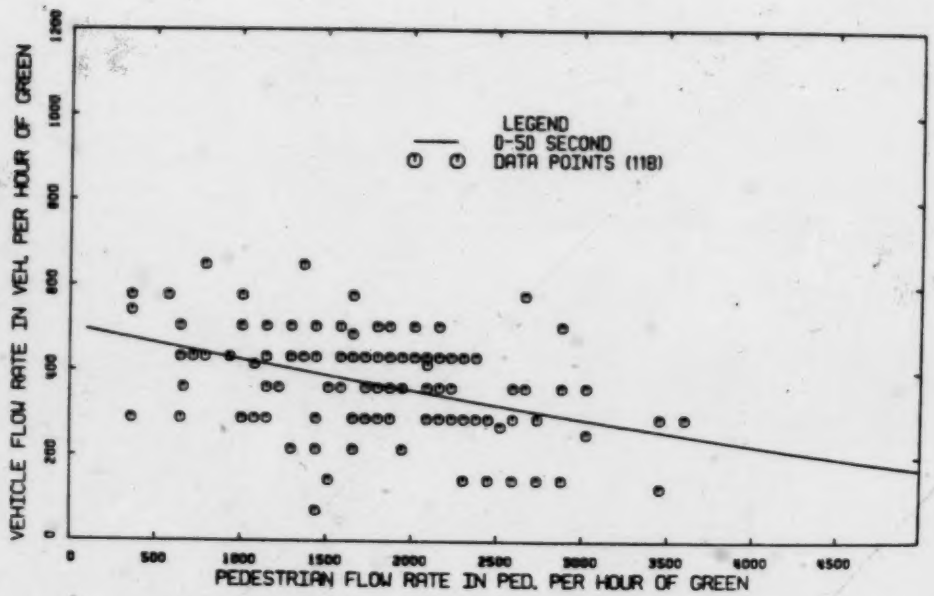


Figure C.22 Location #1; 0-50 sec

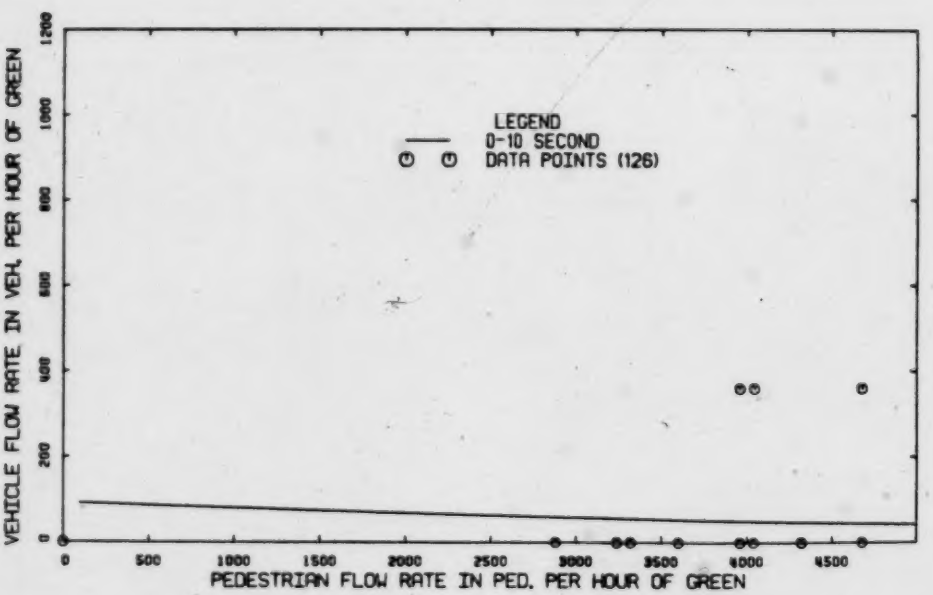


Figure C.23 Location #3; 0-10 sec

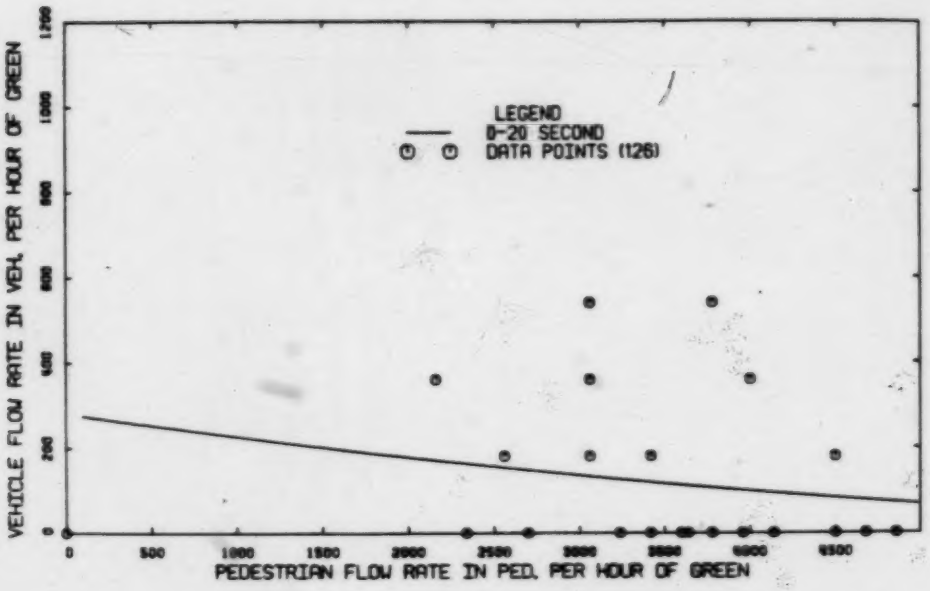


Figure C.24 Location #3; 0-20 sec

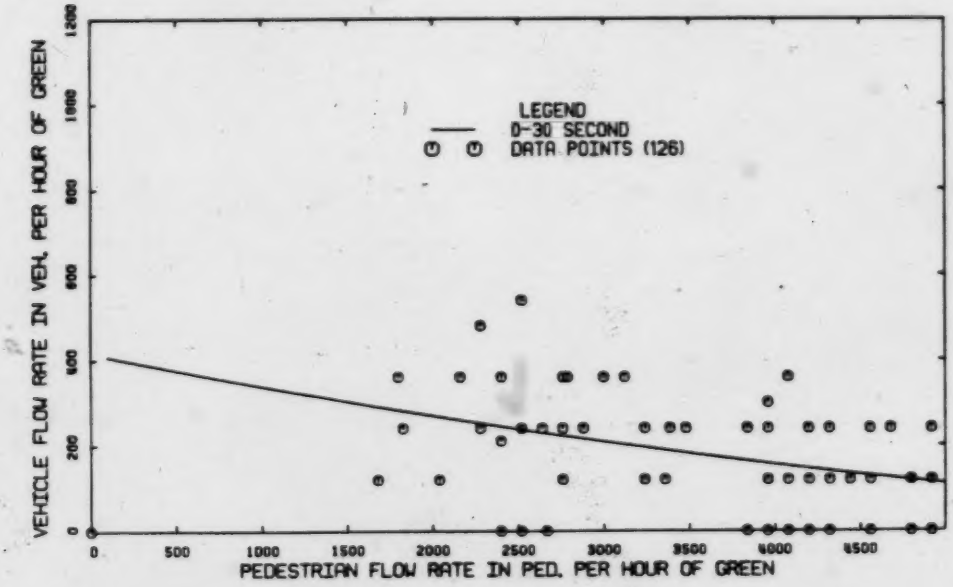


Figure C.25 Location #3; 0-30 sec

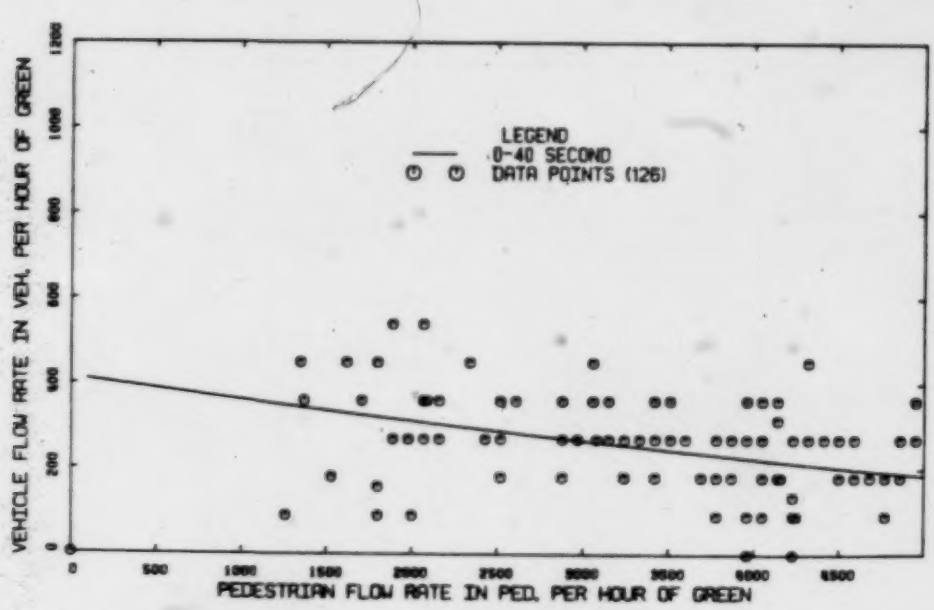


Figure C.26 Location #3; 0-40 sec

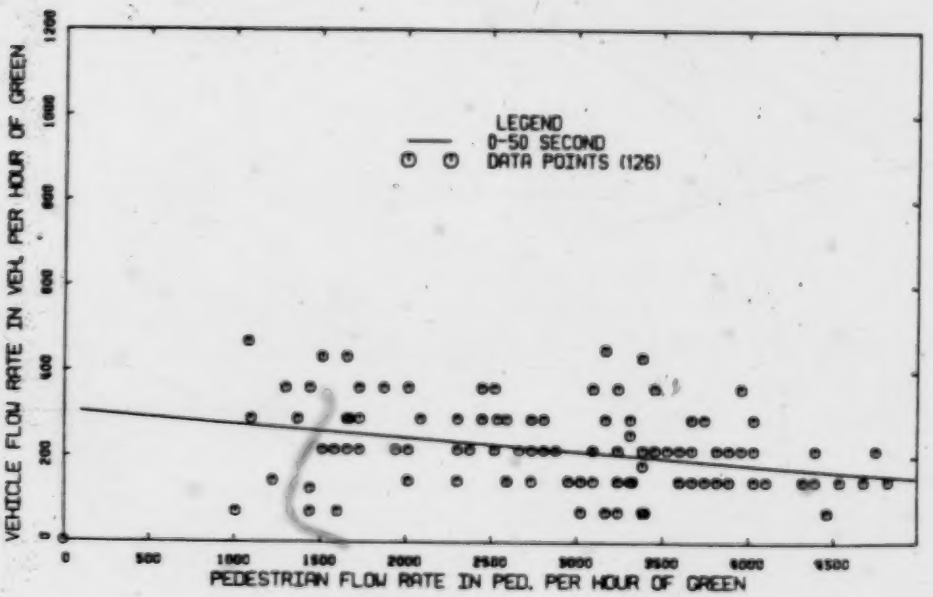


Figure C.27 Location #3; 0-50 sec

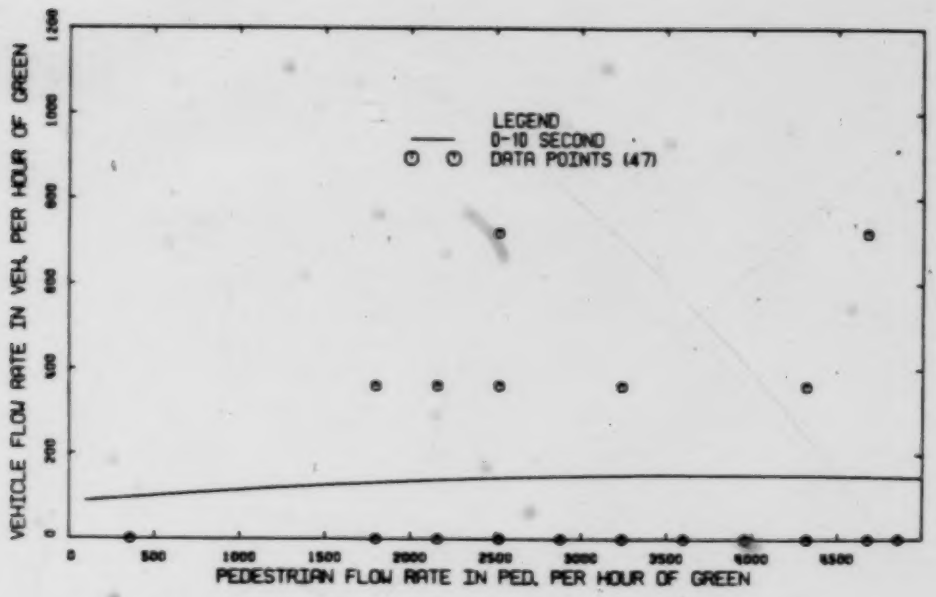


Figure C.28 Location #4; 0-10 sec

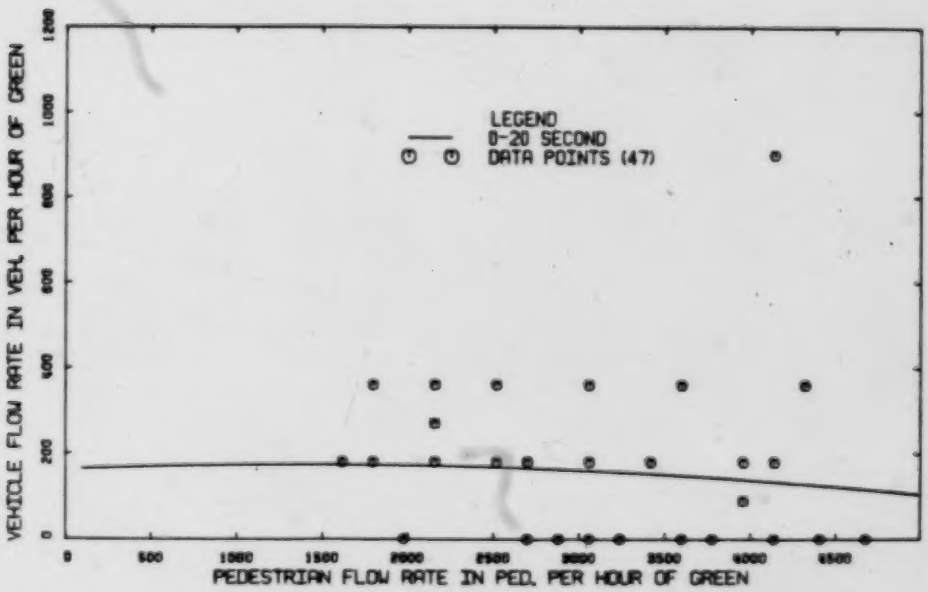


Figure C.29 Location #4; 0-20 sec

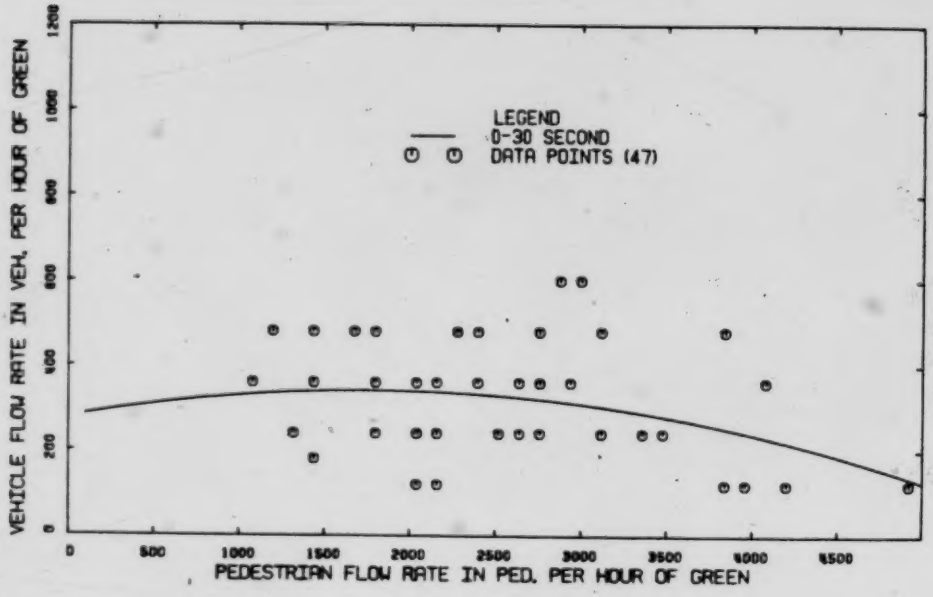


Figure C.30 Location #4; 0-30 sec

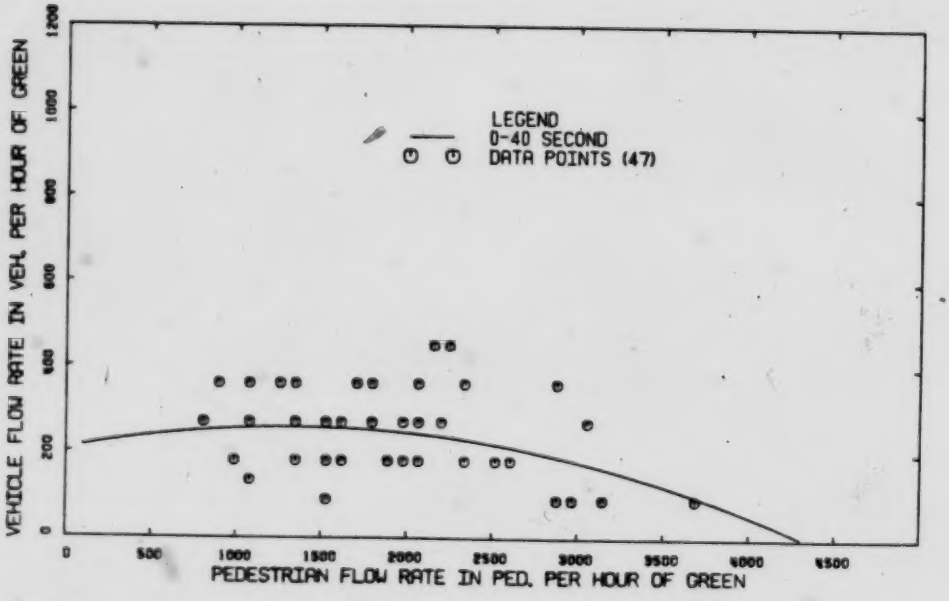


Figure C.31 Location #4; 0-40 sec

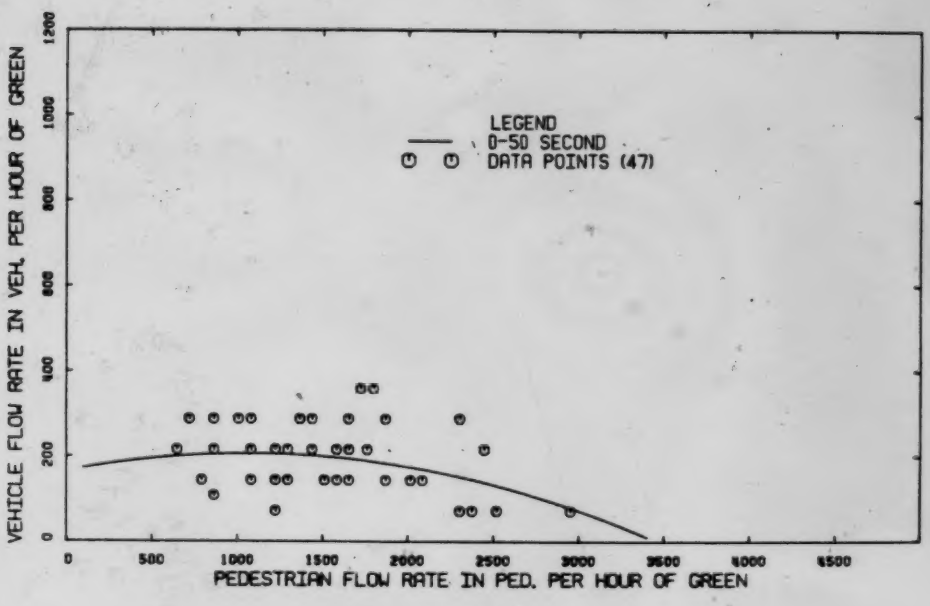


Figure C.32 Location #4; 0-50 sec

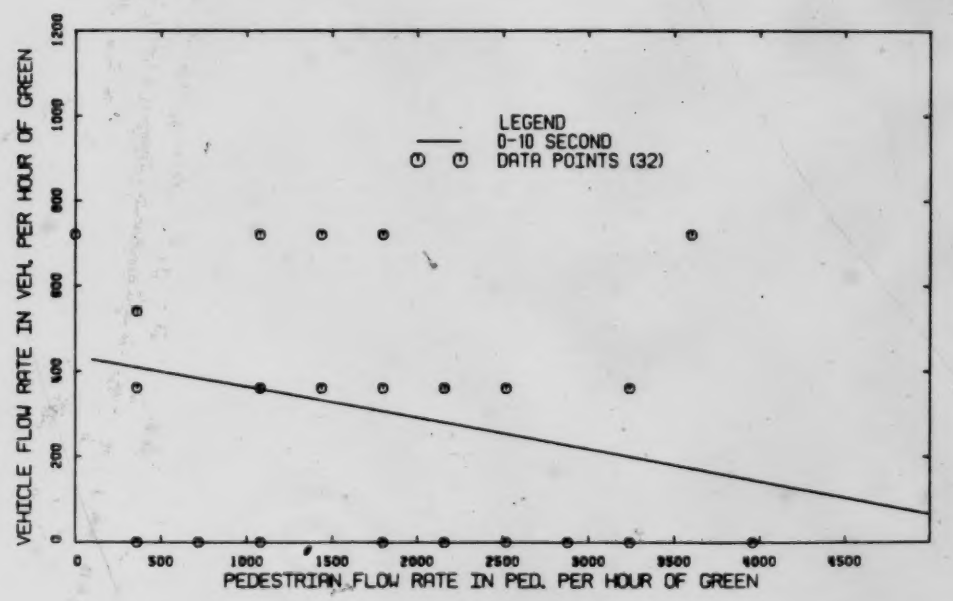


Figure C.33 Location #5; 0-10 sec

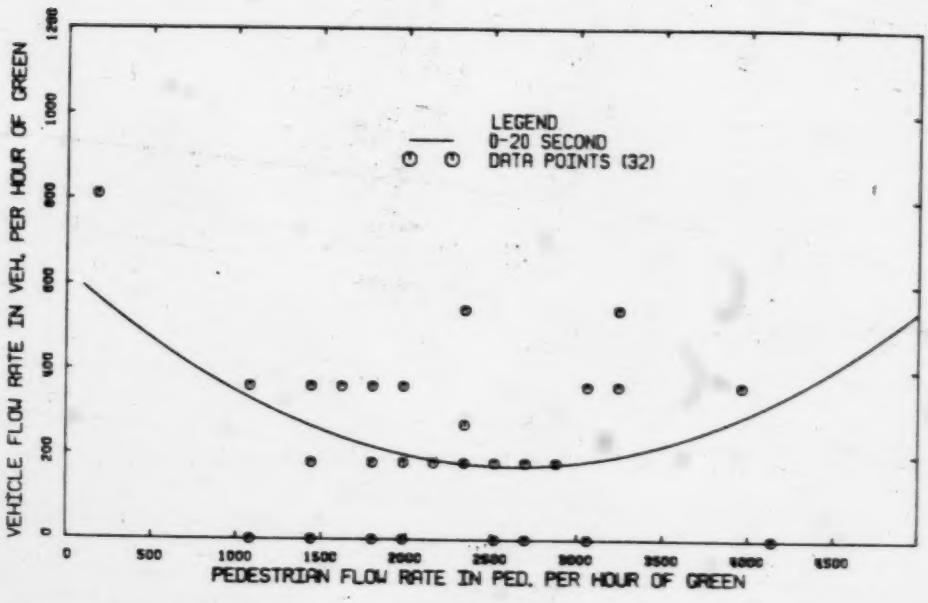


Figure C.34 Location #5; 0-20 sec

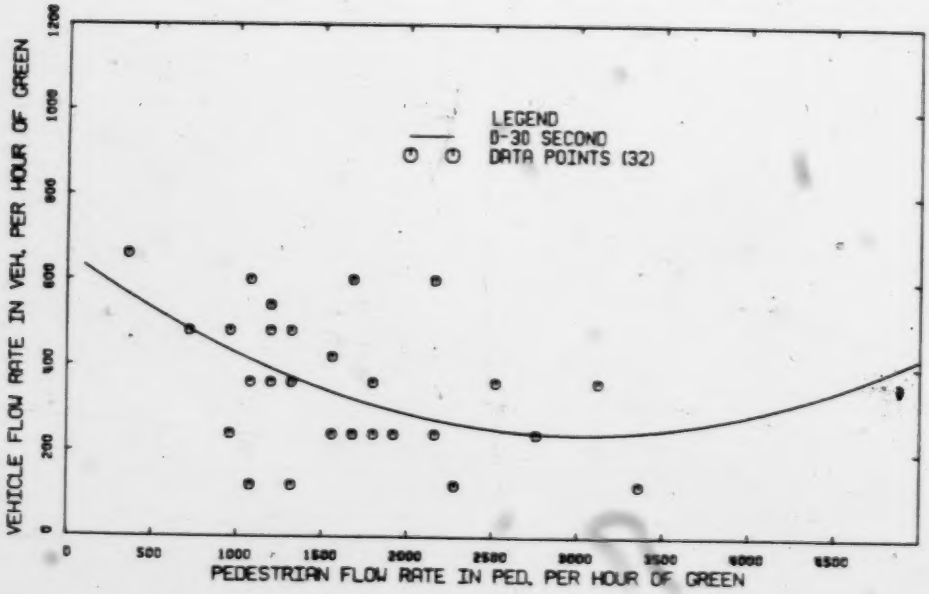


Figure C.35 Location #5; 0-30 sec

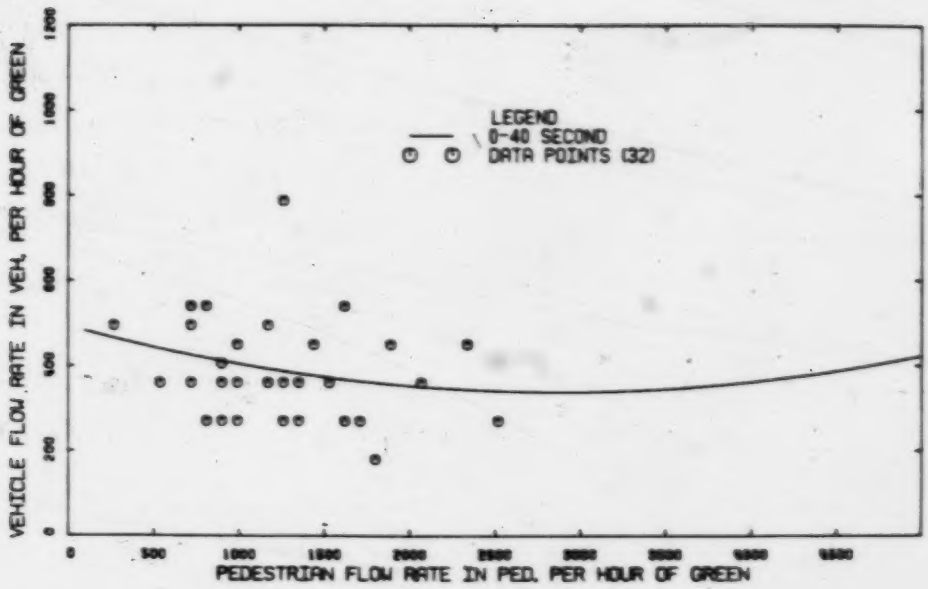


Figure C.36 Location #5; 0-40 sec

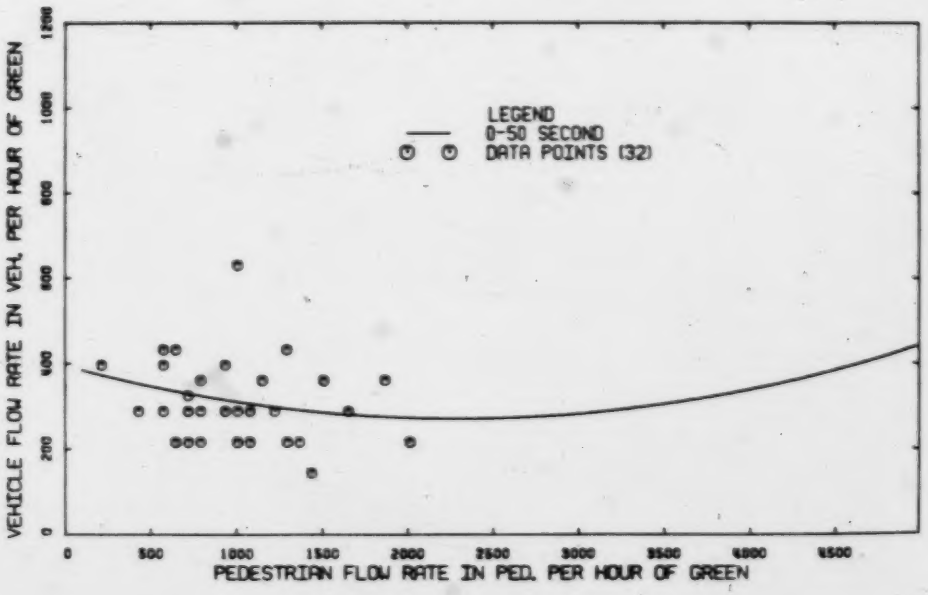


Figure C.37 Location #5; 0-50 sec

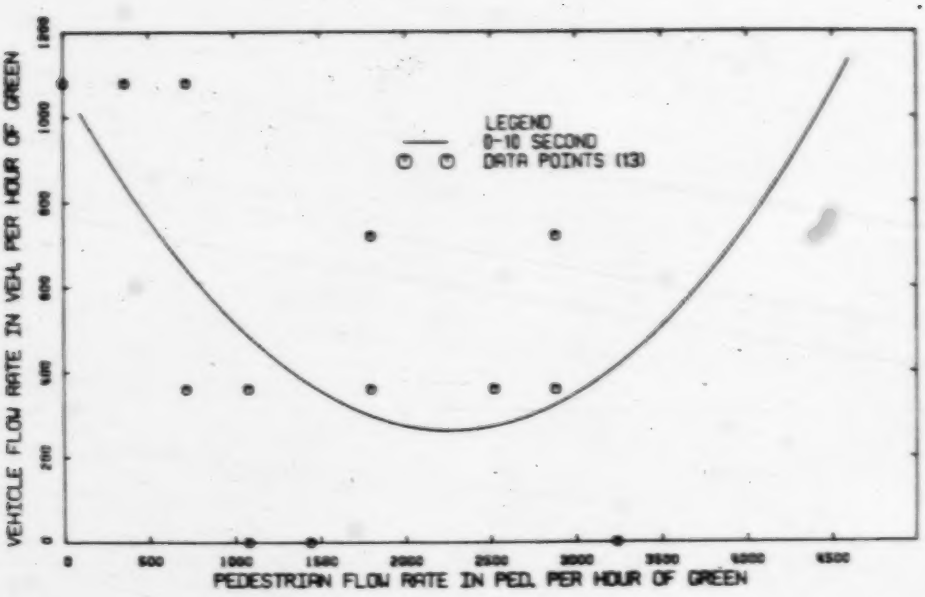


Figure C.38 Location #6; 0-10 sec

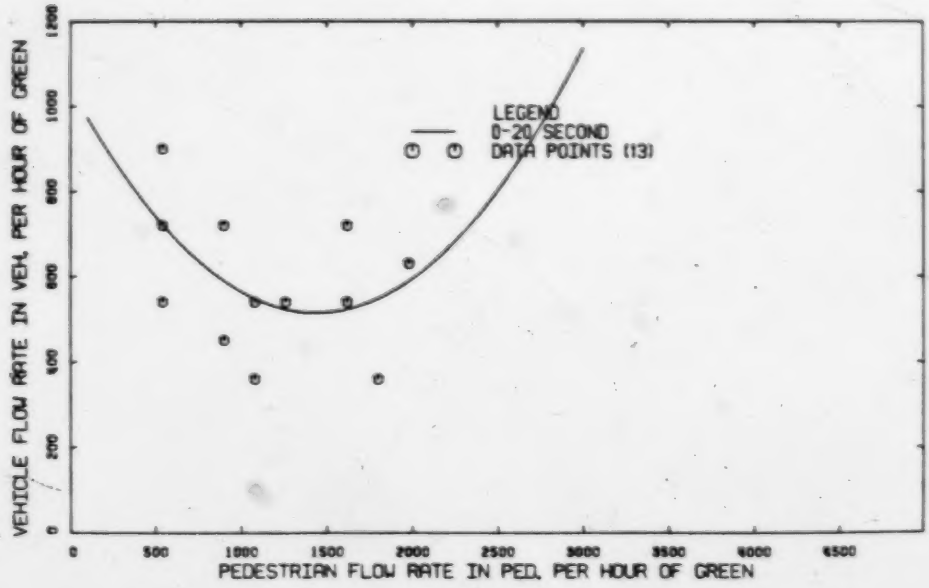


Figure C.39 Location #6; 0-20 sec

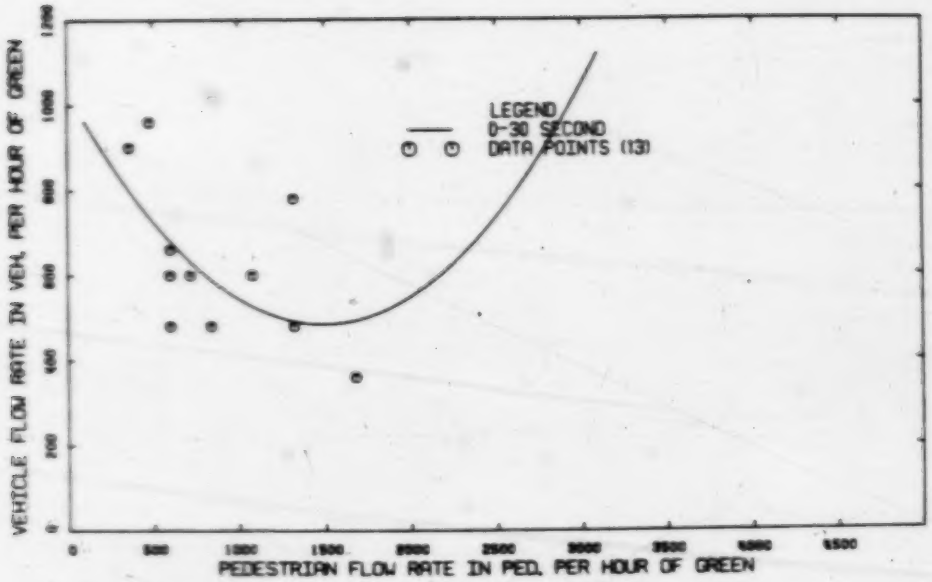


Figure C.40 Location #6; 0-30 sec

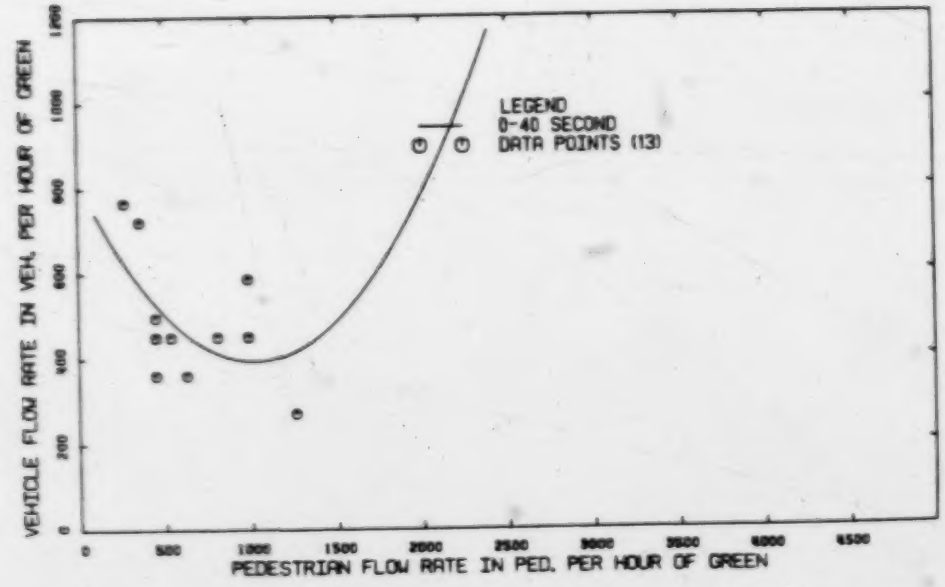


Figure C.41 Location #6; 0-40 sec

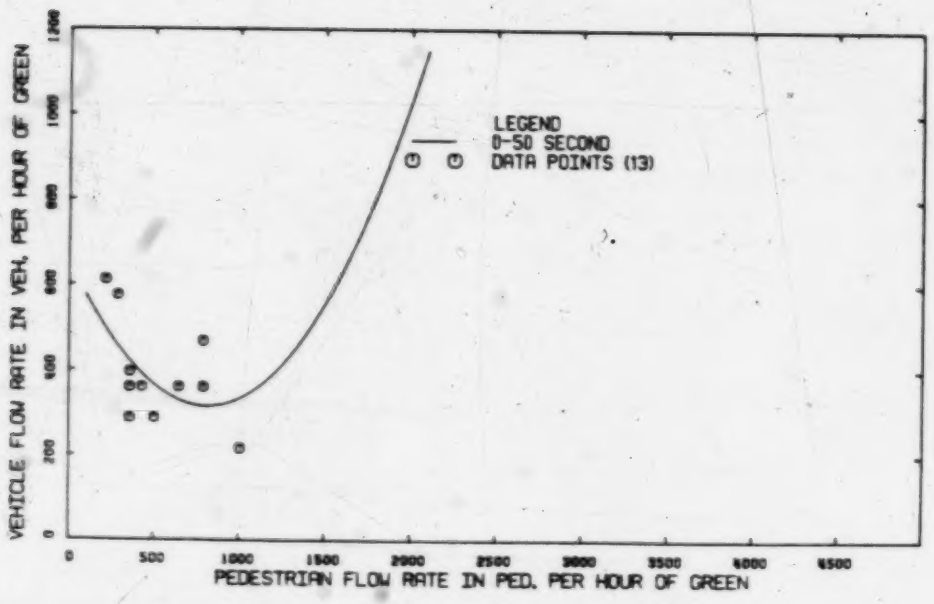


Figure C.42 Location #6; 0-50 sec

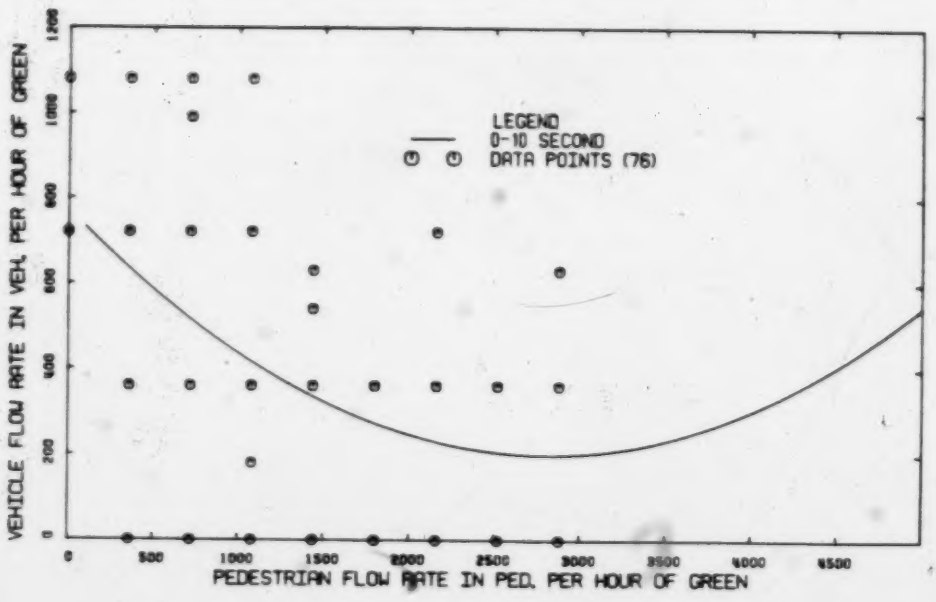


Figure C.43 Location #7; 0-10 sec

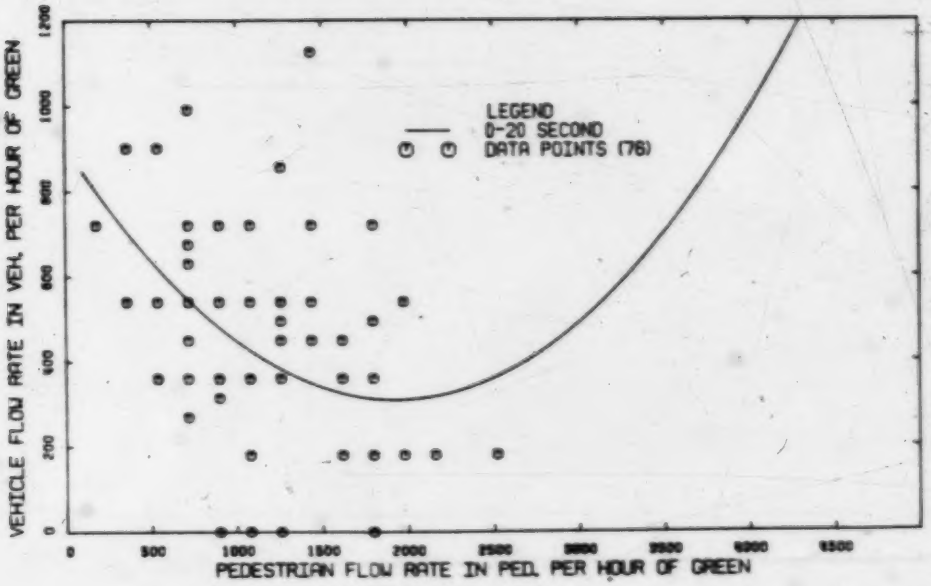


Figure C.44 Location #7; 0-20 sec

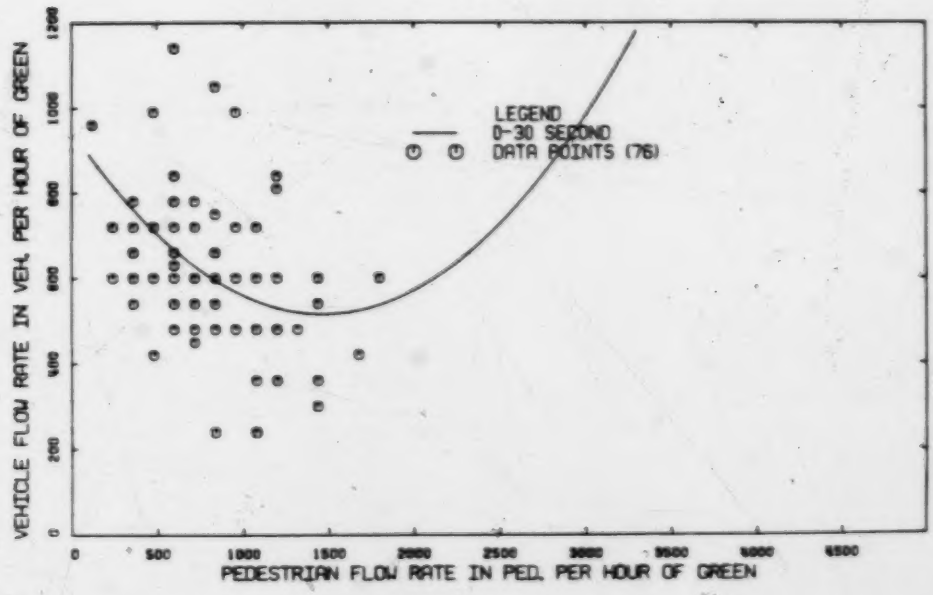


Figure C.45 Location #7; 0-30 sec

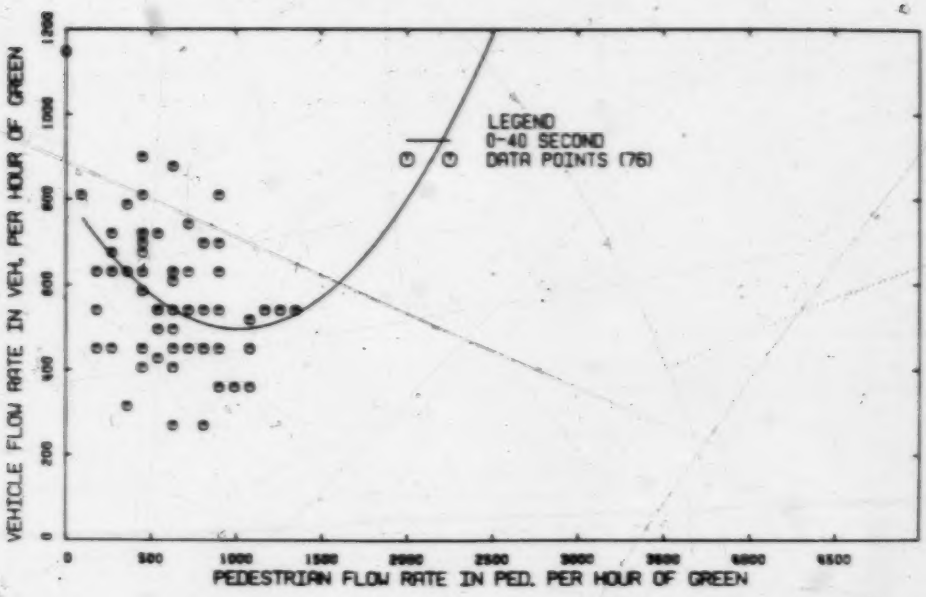


Figure C.46 Location #7; 0-40 sec

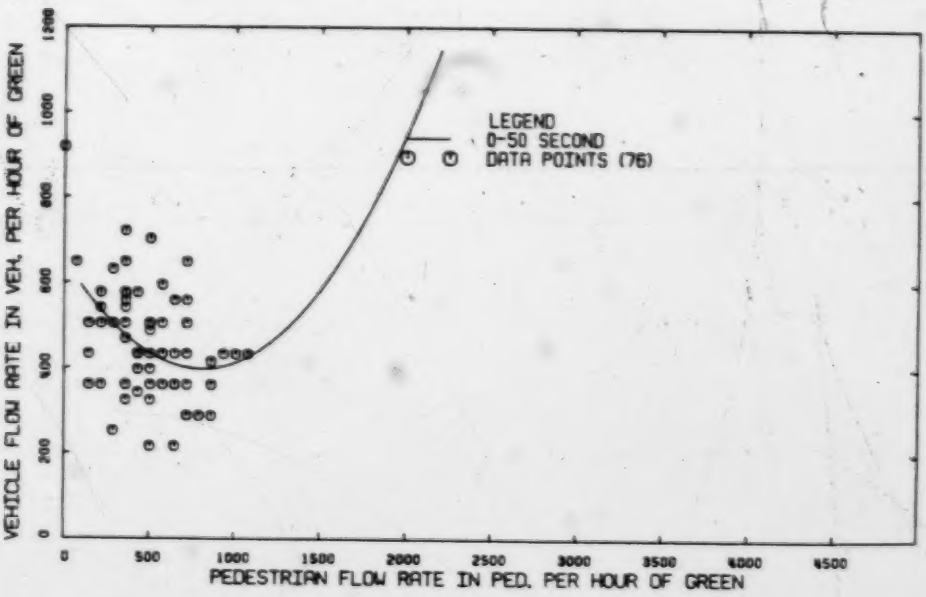


Figure C.47 Location #7; 0-50 sec

(Figures C.48-C.54).

C.8 Flow adjustment factor

Table C-2 was developed from the comparison between all the locations. It is based on observation of flow according to the difference between green and walk (Δ). Presented data were later examined with regard to the other factors such as street width etc. Note: Inconsistencies of the results in the table came from observations made on real flows at locations where other factors were also involved (like radius, pressure etc.) In some cases exceptions were observed as well.

C.9 Results from program HIS

Table C-3 presents the summarized outputs from the computer program HIS.

C.10 Data from all locations

Figures C.55 to C.58 present the theoretical values of the saturation flow measured in 620 observations. The maximum was based on 30 or 40 second intervals according to the group of data. The hidden error can be expected in individual cycles. In the example the maximum cumulative group flow was averaged and found for a 40 second time slice. Individual cycles, in some cases, yielded the maximum flow for 20 seconds. However this was an extreme example, as individual cycles influenced the average and

LOCATION #	INTERVAL	LINEAR REGRESSION	TORONTO FORMULA
1	0-10	Y= 266.61 - 0.0342 x	Y=1073-021x
	0-20	Y= 314.3 - 0.0545 x	
	0-30	Y= 424.3 - 0.0707 x	
	0-40	Y= 549.4 - 0.0734 x	
	0-50	Y= 495.5 - 0.0701 x	
2	0-10	Y= 600. - 0.1 x	
	0-20	Y= 650. - 0.15 x	
	0-30	Y= 800. - 0.22 x	
	0-40	Y= 820. - 0.22 x	
	0-50	Y= 750. - 0.22 x	
3	0-10	Y= 49.77 - 0.0021 x	
	0-20	Y= 146.1 - 0.0151 x	
	0-30	Y= 308.1 - 0.0369 x	
	0-40	Y= 388.6 - 0.0402 x	
	0-50	Y= 307.1 - 0.0312 x	
4	0-10	Y= 175.46 - 0.0095 x	
	0-20	Y= 242.7 - 0.0285 x	
	0-30	Y= 420.3 - 0.0424 x	
	0-40	Y= 319.5 - 0.0436 x	
	0-50	Y= 255.6 - 0.0436 x	
5	0-10	Y= 438.6 - 0.7412 x	
	0-20	Y= 333.5 - 0.047 x	
	0-30	Y= 522.2 - 0.1062 x	
	0-40	Y= 462.1 - 0.0555 x	
	0-50	Y= 369.7 - 0.0555 x	
6	0-10	Y= 793.97 - 0.1878 x	
	0-20	Y= 726.2 - 0.1153 x	
	0-30	Y= 831.1 - 0.2451 x	
	0-40	Y= 630.6 - 0.2249 x	
	0-50	Y= 504.5 - 0.2249 x	
7	0-10	Y= 672.39 - 0.2032 x	
	0-20	Y= 727.8 - 0.2426 x	
	0-30	Y= 823.3 - 0.2401 x	
	0-40	Y= 703.5 - 0.2185 x	
	0-50	Y= 562.8 - 0.2185 x	

Table C-1 Linear regressions for all the seven locations

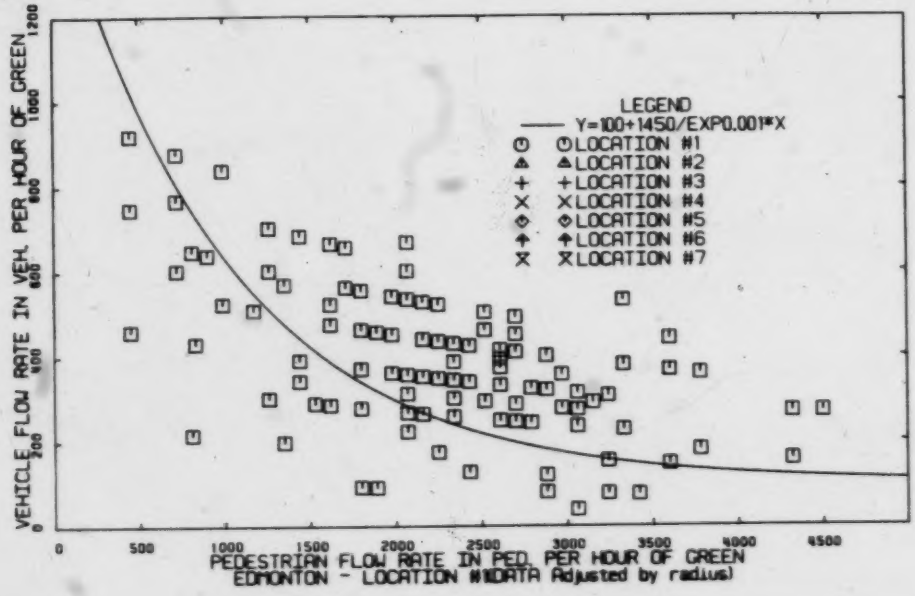


Figure C.48 Location #1 - Adjusted by radius

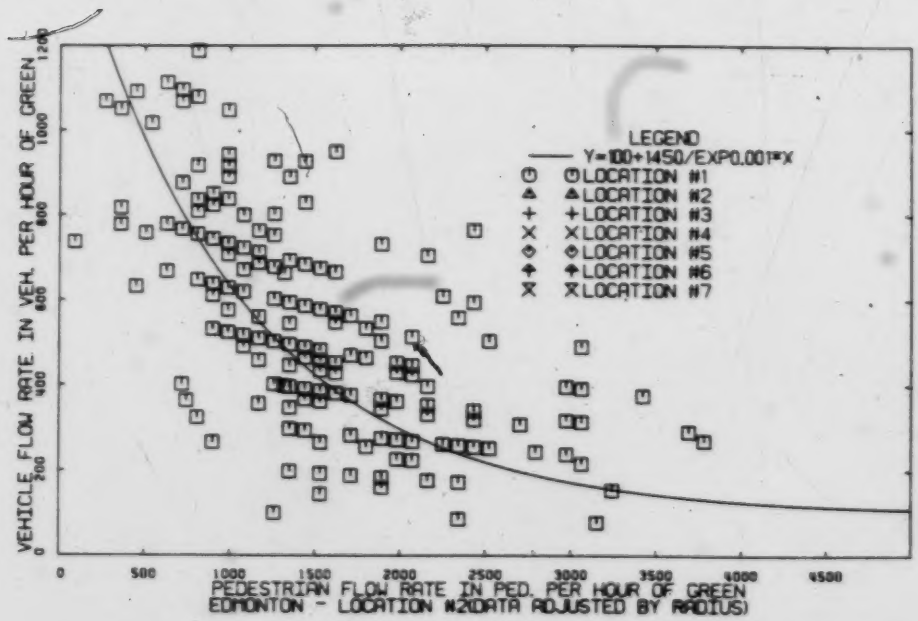


Figure C.49 Location #2 - adjusted by radius

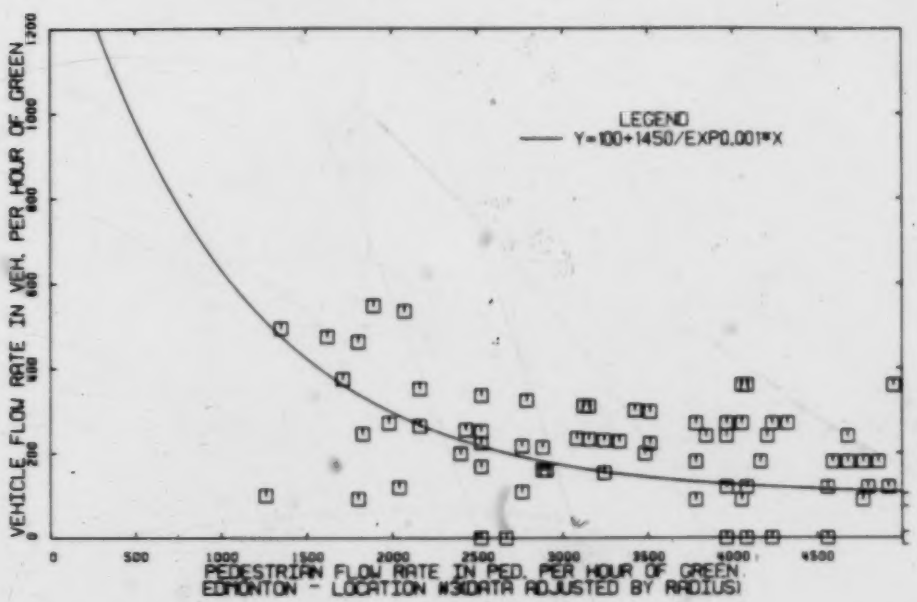

VEHICLE FLOW RATE IN VEH. PER HOUR OF GREEN
LEGEND
Y=100+1450/EXP0.001*X
PEDESTRIAN FLOW RATE IN PED. PER HOUR OF GREEN
EDMONTON - LOCATION #3(DATA ADJUSTED BY RADIUS)

Figure C.50 Location #3 - adjusted by radius

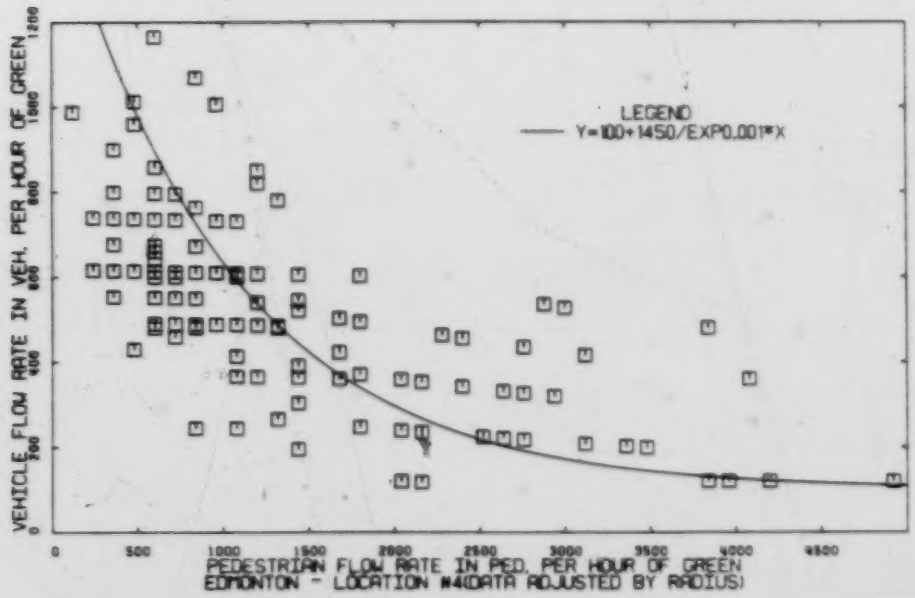

VEHICLE FLOW RATE IN VEH. PER HOUR OF GREEN
LEGEND
Y=100+1450/EXP0.001*X
PEDESTRIAN FLOW RATE IN PED. PER HOUR OF GREEN
EDMONTON - LOCATION #4(DATA ADJUSTED BY RADIUS)

Figure C.51 Location #4 - adjusted by radius

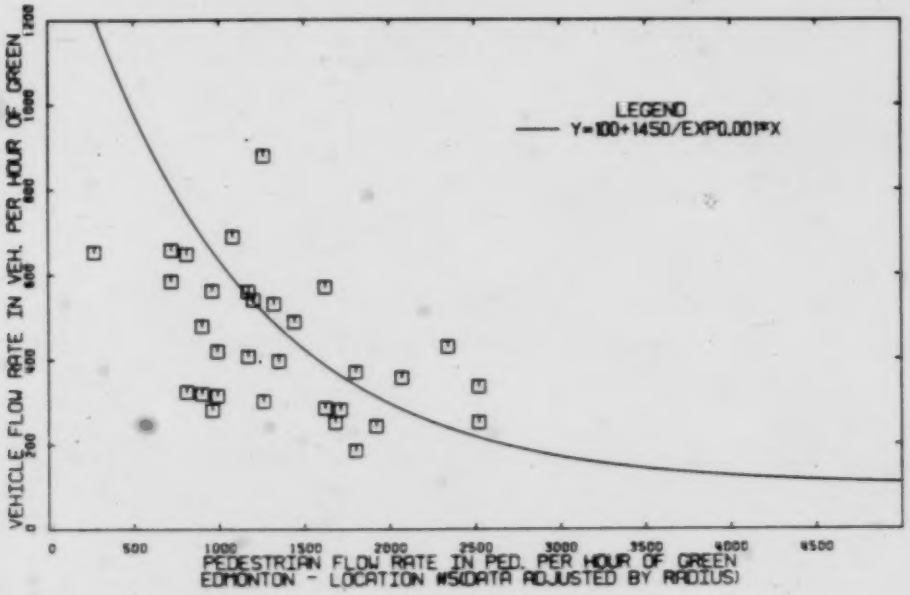


Figure C.52 Location #5 - adjusted by radius

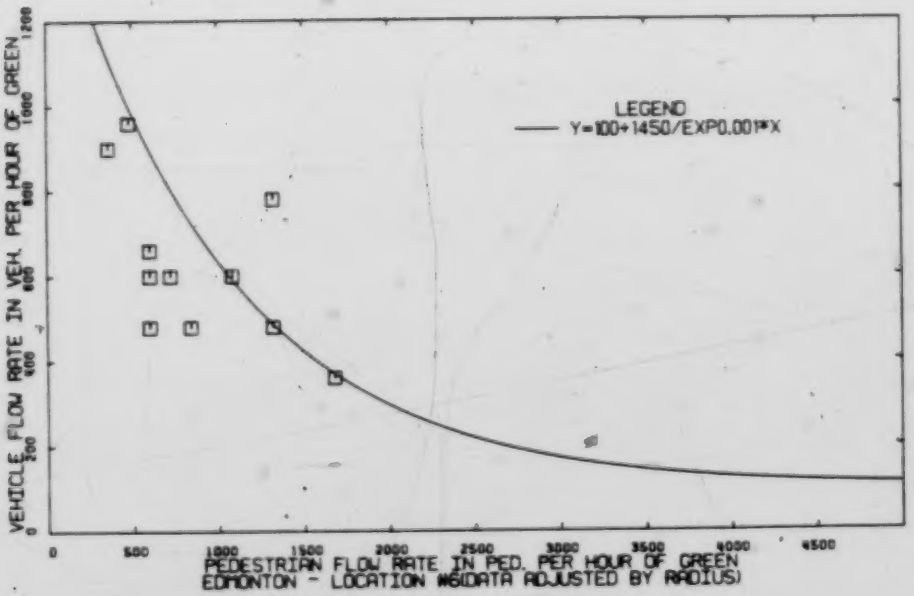


Figure C.53 Location #6 - adjusted by radius

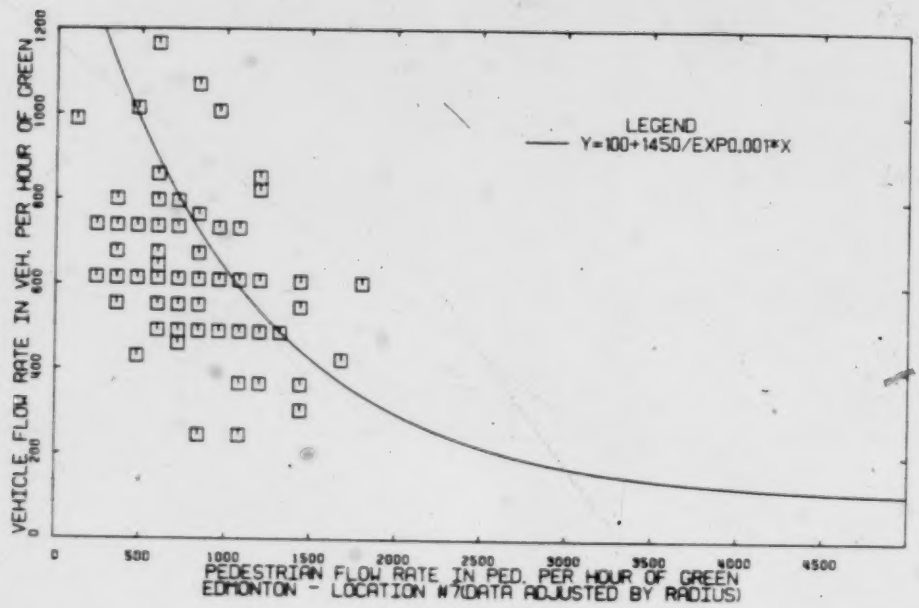


Figure C.54 Location #7 - adjusted by radius

GREEN-WALK DIFFEREN. (GgD)	PEDESTRIAN FLOW RANGE					
	0	500	1000	1500	2000	3000
	SUBSTRACTED FLOW					
14 sec	100	50-0	100-50	50-0	0	0
13 sec	300	300-100	100-50	50-0	0	0
12 sec	300	300-200	200-50	50-0	0	0
11 sec	350	300	220-50	50-0	50-0	0
10 sec	400	300	200-100	150-0	50-0	0
<10 sec	400	400-350	350-200	200-100	100-50	50-25

Table C-2 Rough values for the substraction from flows calculated by the basic formula (derived from field observations)

Table C-3 Results from the program HIS

LOCATION (GREEN TIME)	# OF VEHICLES ; VEHICLE FLOW RATE ; PEDESTRIAN FLOW RATE					# OF OBSERV.
	0-10	10-20	20-30	30-40	40-50	
#1 (39)	0.26 93.9 4564.7	0.18 63.5 3780.	1.1 397.1 1265.3	2.5 888.7 455.3	0.88 315.0 26.5	68
#1 (39)	0.27 96.9 3586.2	0.31 110.8 2568.5	1.19 429.2 962.3	2.17 782.3 360.0	1.1 394.6 0.0	26
#1 (39)	0.43 154.3 5451.4	0.29 102.9 2957.1	1.57 565.7 900.0	2.61 938.6 154.3	0.39 141.4 0.0	14
#1 (39)	0.8 288 1836	1.15 414.0 756.0	1.9 684.0 252.0	2.35 846.0 360.0	0.75 270.0 36.0	10
#2 (33)	1.29 465.0 1166.7	0.5 180.0 1140.0	2.08 750.0 180.0	0.17 60.0 0.0	0 0 0	6
#2 (33)	0.83 300 2880	1.08 390 1260	1.83 660 360	2.04 735.0 300	0.17 60.0 0	6
#2 (33)	0.77 278.2 2127.3	0.84 302.7 1505.6	1.69 609.6 540.0	2.31 830.5 310.9	1.81 650.5 130.9	22
#2 (40)	0.38 135.6 3930.4	0.3 108.5 2993.7	1.64 590.5 769.3	2.45 880.3 14.8	0 0 0	73
#2 (40)	0.82 295.2 3088.8	0.87 313.2 1965.6	1.71 615.6 784.8	2.42 871.2 266.4	1.7 612.0 0.0	50
#2 (40)	1.09 390.9 2211.4	0.96 347.1 1604.6	2.08 748.3 504.0	2.61 941.1 277.7	1.55 558 20.6	35
#2 (40)	0.53 190.6 3007.1	0.59 211.8 2287.1	1.62 582.4 804.7	1.93 693.6 508.2	1.34 481.8 42.4	17
#3 (31)	0.08 27.3 8629.7	0.08 27.3 4872.1	0.78 279.6 1663.6	1.73 624.5 201.8	0.03 10.9 0.0	66

Table C-3 Continued

LOCATION (GREEN TIME)	# OF VEHICLES; VEHICLE FLOW RATE; PEDESTRIAN FLOW RATE					# OF OBSERV.
	0-10	10-20	20-30	30-40	40-50	
#3 (31)	0.13 45.0 7931.3	0.19 67.5 3818.8	0.72 258.8 15.8	1.66 596.3 101.3	0 0 0	32
#3 (40)	0.07 26.7 9136.3	0.35 126.7 4162.9	0.87 313.3 1720.0	1.02 366.7 1533.3	1.45 523.3 53.3	27
#4 (22)	0.36 130.2 4752.8	0.4 145.5 2611.9	1.84 662.3 245.1	0.02 7.6 0.0	0 0 0	47
#5 (31)	0.33 120.0 1680.0	0.67 240.0 1560.0	2.67 960 480	0 0 0	0 0 0	3
#5 (31)	0.89 319.1 2110.9	0.43 155.5 2585.6	1.39 144.1 572.7	1.63 585.0 16.4	0 0 0	22
#5 (31)	0.71 257.1 2005.7	0.57 205.7 1902.9	1.86 668.6 565.7	1.64 591.4 51.4	0 0 0	7
#6 (30)	1.38 498.5 1578.5	1.92 692.3 692.3	1.81 650.7 387.7	0.23 83.1 0.0	0 0 0	13
#7	1.28 460.7 1285.7	1.18 426.4 1011.4	2.39 859.3 214.3	1.05 379.3 17.1	0 0 0	42
#7	0.99 357.4 1249.4	1.68 603.5 847.1	3.06 1101.1 222.4	1.14 410.3 10.6	0 0 0	34

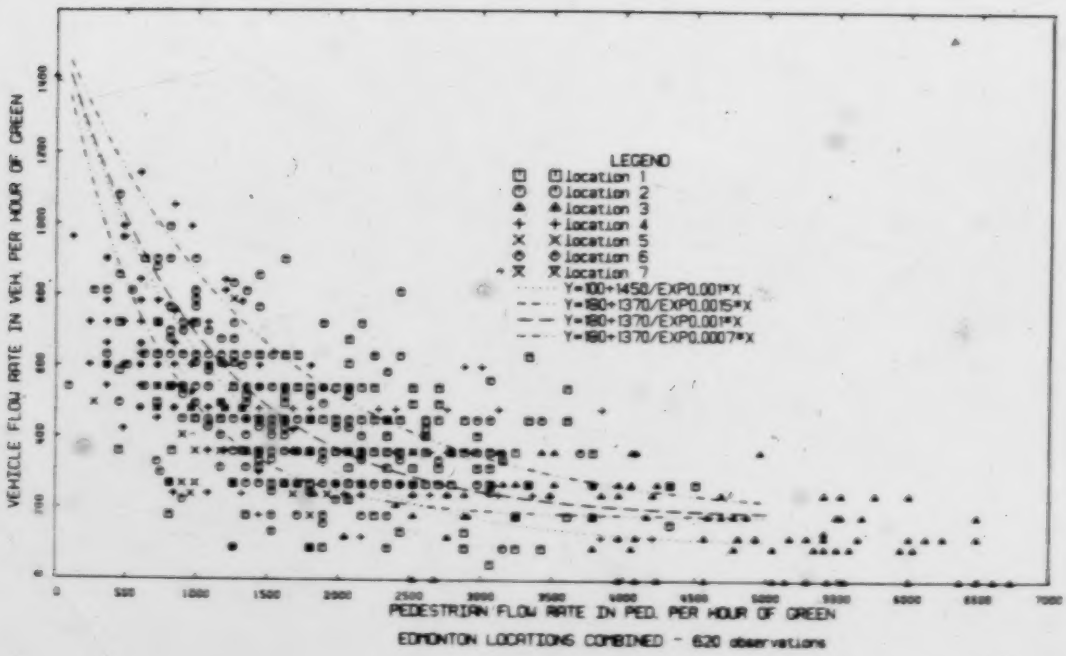


Figure C.55 Presentation of 620 observed data from seven locations

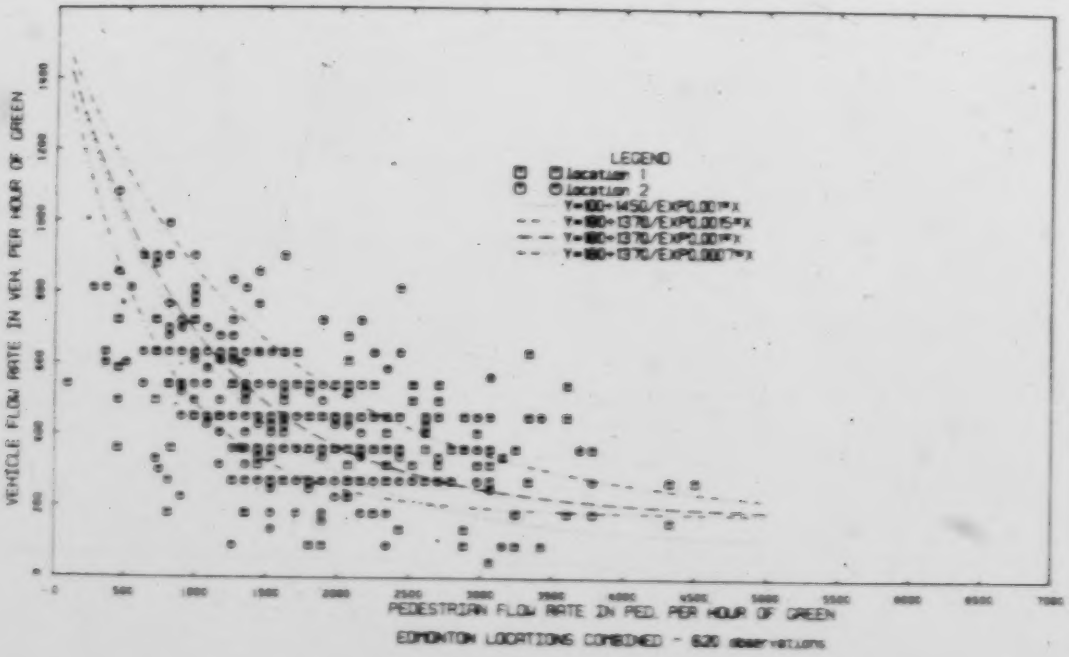


Figure C.56 Data observed at locations #1 and #2

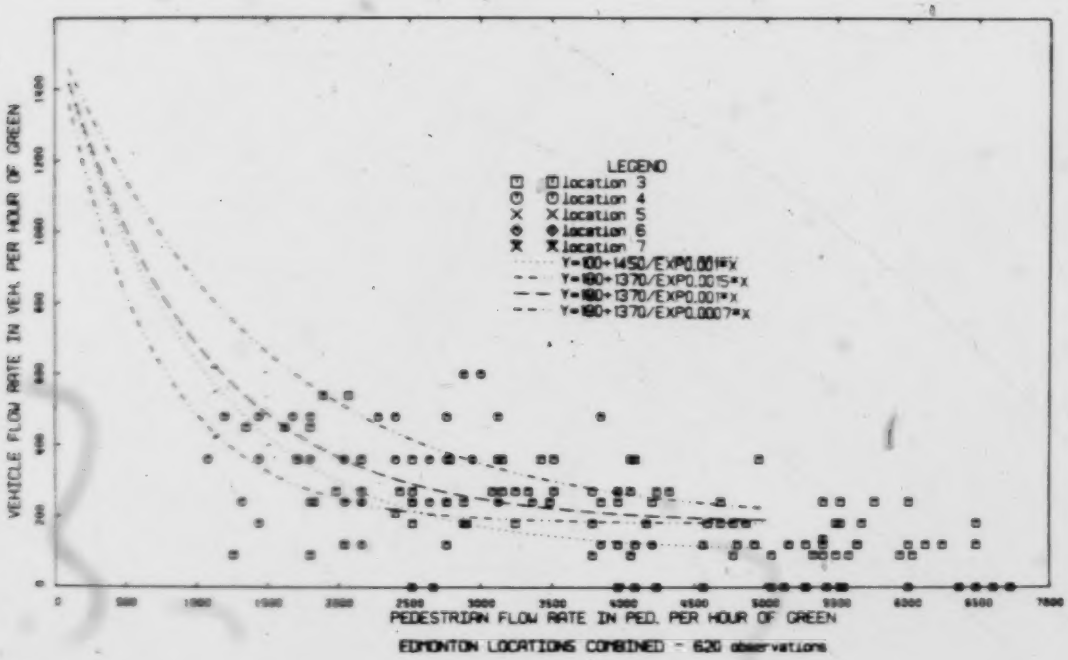


Figure C.57 Data observed at locations #3 and #4

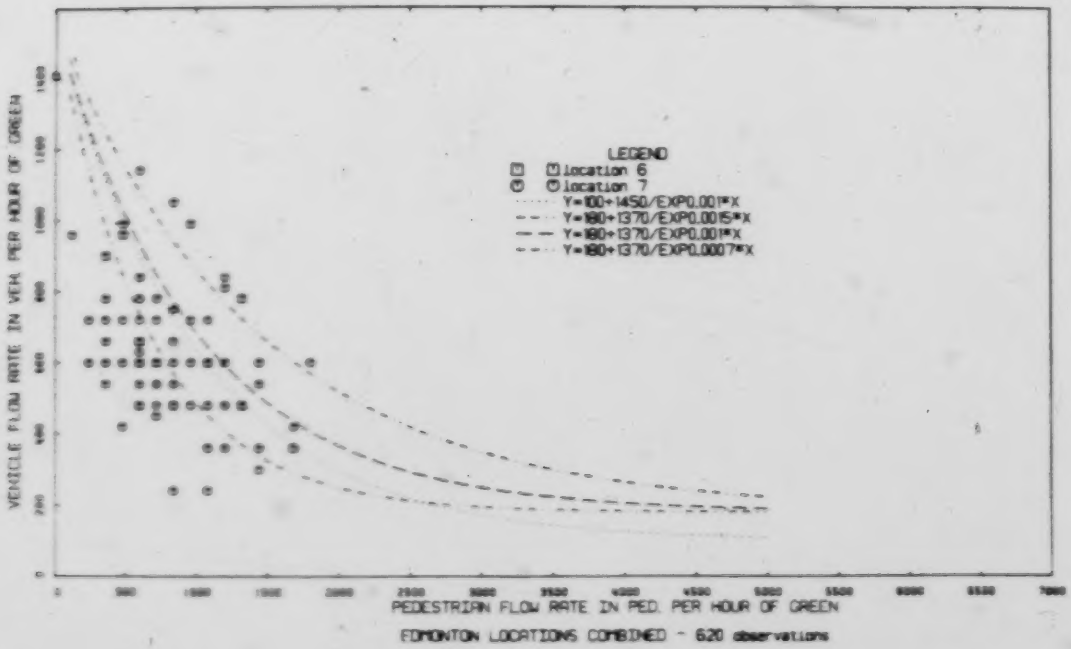


Figure C.58 Data observed at locations #6 and #7

this type of error can be neglected. Figures are presented to show the total of 620 observations as well as the data from locations with similar conditions.

C.11 Statistical evaluation

Figure C.59 presents the example of output from statistical program HIST.

Table C-4 contains the statistical values obtained from program HIST. Table C-5 contains the results from the calculation of upper and lower limits for vehicular flow mean.

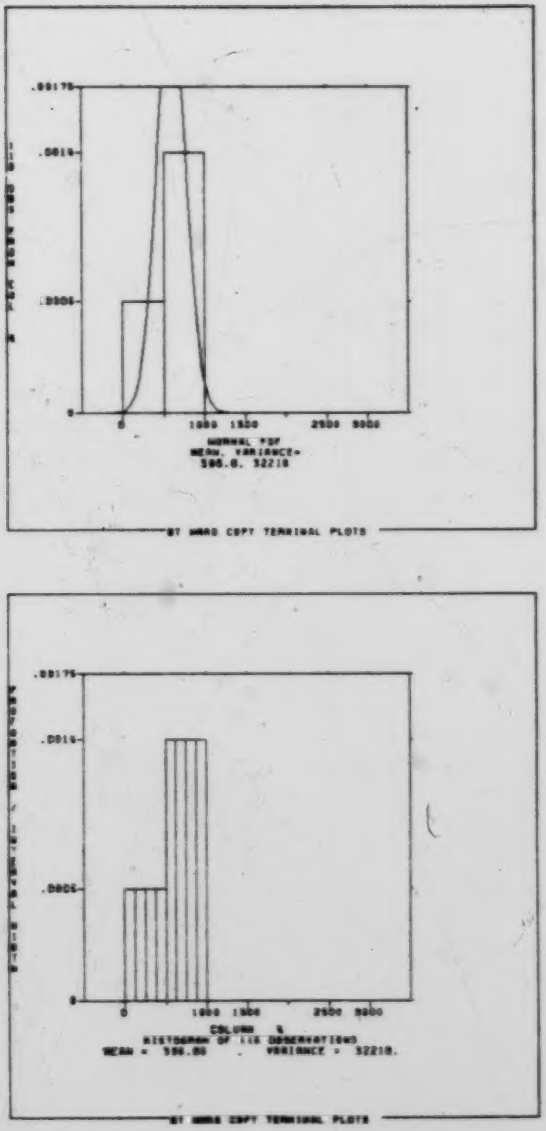


Figure C.59 Example of the output from the statistical program HIST

CATEGORY	# OF OBSER.	COLUMN 1		COLUMN 2		COLUMN 4		CHI
		$\bar{x}$	V_x	$\bar{x}_2$	$V_{\bar{x}_2}$	$\bar{x}_4$	$V_{\bar{x}_4}$	D.F
0 - 800	25	350.69	15 628	720	48 279	999.9	58 273	999
800 - 1000	110	799.76	19 406	609	23 602	803.5	41 719	804
1000 - 1500	110	1266.5	15 595	489	26 277	596.8	32 218	597
1500 - 2000	98	1748.8	24 159	387	20 114	417.1	22 730	417
2000 - 2500	78	2213.1	17 718	371	23 834	360.2	22 266	360
2500 - 3000	57	2714.4	24 534	327	17 816	256.8	14 416	
3000 - 3500	34	3203.5	20 133	308	19 859	264.1	14 813	
3500 - 4000	23	3726.6	20 800	840	19 718	222.9	15 057	
4000 - 4500	18	4177.8	16 135	207	12 967	207.1	12 965	
4500 - 5000	15	4744.0	20 534	160	7 157	160	7 157	
5000 - 5500	19	5291.9	18 534	73	5 322	73	5 322	
5500 - 6000	15	5707.8	34 235	124	6 666	124	6 797	
6000 - 6500	11	6242.7	47 682	90	6 797	90	6 666	
6500 - 7000	3	6680.0	48 800	0	0	0	0	
7000 - 9500	5	8573.3	0.96 E+06	0	0	0	0	

Table C-4 Results from the computer program HIST

CATEGORY	COLUMN 2			COLUMN 4				
	$\bar{x}_2$	t	k_{95}	$\bar{x}_4$	t	k_{95}	UL	LL
0 - 800	720	220	95.7	1000	241.4	87.9	1088	912
800 - 1000	609	183	34.2	804	204.9	38.3	842	766
1000 - 1500	489	162	30.3	597	179.5	33.5	631	563
1500 - 2000	387	142	28.1	412	150.7	29.9	442	382
2000 - 2500	271	155	34.3	360	149	33.1	393	327
2500 - 3000	327	132	34.3	297	120	31.2	328	266
3000 - 3500	308	141	24.2	266	121.7	20.1	286	246
3500 - 4000	240	140	23.2	223	122.7	23.5	249	197
4000 - 4500	207	114	52.7	207	113.9	52.6	260	154
4500 - 5000	160	85	22.	160	84.6	42.8	203	117
5000 - 5500	74	73	32.8	73	73	32.8	106	40
5500 - 6000	124	82	21.2	124	82.4	41.7	166	82
6000 - 6500	90	82	24.7	90	81.6	48.2	138	42
6500 - 7000	0	0	0	0	0	0	0	0
7000 - 9500	0	0	0	0	0	0	0	0

Table C-5 Upper and lower limits for mean at 95% confidence level

APPENDIX D - ILLUSTRATIVE EXAMPLE

Illustrative example of the evaluation procedure is presented in this Appendix in the following steps:

D.1. Input data

- street width W (m)
- turning radius R (m)
- cycle time C (sec)
- green time G (sec)
- pedestrian walk g (sec)
- number of pedestrians entering the interaction zone from both sides in one hour based on 30 min observation

$$*N = N_n + N_f \text{ (pedestrians/30 minutes)} \qquad (26)$$

- observed traffic behaviour (pressure, obstructions, etc.)

D.2. Decision

- decide whether the intersection conditions are typical
- YES → continue the procedure
- NO → adjust procedure or use other principles

D.3. Pedestrian flow rate

- calculate pedestrian flow rate

$$N = 2(*N) \quad \text{(pedestrians/h)} \qquad (27)$$

$$n = \frac{3600}{C} \quad \text{(number of cycles/h)} \qquad (28)$$

$$+N = \frac{N}{n} \quad \text{(number of pedestrians/cycle)} \qquad (29)$$

$$P = \frac{+N}{35} * 3600 \quad \text{(pedestrians/h green)} \qquad (30)$$

based on average time slice with highest saturation flow

D.4. Decision

pedestrian flow rate belongs to region:

A. 0-2000 - apply adjustment factors

B. 2000-5000 - read vehicle flow rate directly from graph or formula (Figure 8.4)

C. 5000-more - use 170 pcu/h green for 5000 ped/h green with linear interpolation ending at 50 pcu/h green for 8000 ped/h green

OR use 1.5-1.0 vehicle/cycle with the diminishing tendency

*OUTPUT for Case C

D.5. Region 0-5000 ped/h green

Case B and A

- calculate vehicle flow rate

$$S_p = 170 + 1380/e^{0.001*P} \quad (24)$$

where: P - pedestrian flow rate (pedestrians/h green)

S_p - vehicle flow rate (pcu/h green)

- or use graph (Figure 8.4)

D.6. Decision

- adjust the vehicle flow rate ± 50 to 100 pcu according to the traffic conditions (based on your experience) if necessary

*OUTPUT for Case B

D.7. Region 0-2000

Case A

A. find radius adjustment factor (F_R)

- use Figure 8.2 according to the turning radius R and the pedestrian flow rate P.

B. find Δ and flow factor (S_F)

- use Figure 8.3 according to the ratio of street width (w) over difference Δ

$$\Delta = G - g \quad (31)$$

where: Δ - difference between green and walk

G - green interval

g - pedestrian walk interval

C. apply factors (F_R & S_F) to the flow

$$S = (S_p - S_F) * F_R \qquad (25)$$

where: S - adjusted right turning saturation flow for a given location (pcu/h green)

S_p - vehicle flow rate from step D.5

S_F - flow adjustment factor (pcu/h green)

F_R - radius adjustment factor < 1.0 (nondimensional)

*OUTPUT for Case A

D.8. Capacity

For the capacity tasks the right turning vehicles penetrating during the red period should be included. Right turns during the red period were not included in this research.